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15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY, DETROI--ETC(U)
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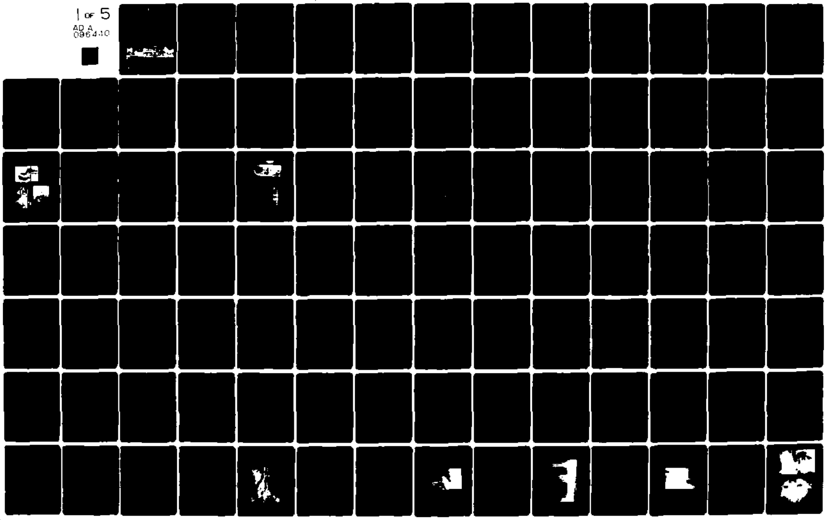

LEVEL II

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TECHNICAL REPORT GL-81-2

15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY DETROIT AREA, MICHIGAN

AD A096440

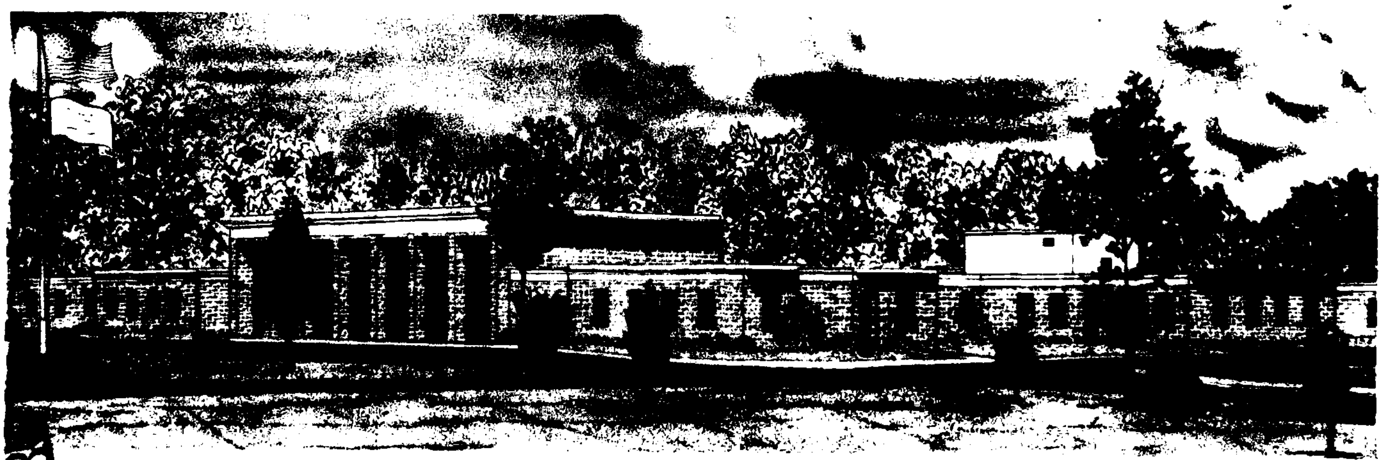
Geotechnical Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

and

U. S. Army Engineer District, Detroit
Detroit, Mich. 48231

January 1981

Final Report



Prepared for Environmental Protection Agency, Region 5
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The study consisted of field and laboratory investigations, construction evaluation, and geotechnical and structural analyses to determine the cause(s) of distress and failure of a 2600-ft section of 12-ft 9-in. diameter concrete-lined sanitary sewer tunnel in the Detroit, Mich., area. The work was performed at the request of the Environmental Protection Agency, Region V, Chicago, Ill. The report includes summaries of all pertinent construction records, results of all pertinent past and current field and laboratory tests

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on construction and geotechnical materials, and detailed geotechnical and structural analyses based on observed conditions and measured parameters. Factors that could have potentially influenced or contributed to the distress were investigated; the findings eliminated certain possible factors and identified the essential causes and mechanisms. The conclusions are limited to those which could be made based on results of the analyses and facts and evidence documented in the report.

The section of tunnel that experienced distress is at a depth of approximately 65 ft and was mined through lake-bed deposits that contain strata of silt and fine sand. The water table is normally approximately 20 ft above the invert but was drawn down for construction. The silts and fine sands are highly susceptible to piping under even small heads. Some of the construction joints in the concrete liner were made without waterstops and concrete placement procedures were such that cold joints occurred in the liner.

The tunnel was completed and placed in service in 1972; however, the distress actually began immediately following construction as soon as the groundwater level was sufficiently high to initiate piping of soil through the open construction and cold joints. This piping took place over a significant period of time. As greater loss of support occurred, the concrete liner deformed, the joints opened wider, and more soil was allowed to pipe into the tunnel. These events progressed until the distress was manifested by the crack pattern found and by total collapse at Distressed Area 1 and partial collapse at Distressed Area 3. Varying degrees of distress were experienced along the 2600-ft section depending upon the location of the strata of piping soil with respect to open construction joints and/or cold joints.

PREFACE

This study was conducted during the period March through August 1980 by the Geotechnical Laboratory (GL) and the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES), and the Construction-Operations Division of the U. S. Army Engineer District, Detroit. The U. S. Army Corps of Engineers (CE) was authorized to perform the study by a letter dated 11 March 1980 from Region V, U. S. Environmental Protection Agency (EPA). The WES portion of the study was authorized by Detroit District's Intra-Army Order for Reimbursable Services No. NCE-IA-80-055 dated 27 March 1980.

The construction portion of this report was prepared by Messrs. D. Albert and B. L[illegible]nce of the Construction-Operations Division, Detroit District. The geotechnical portion of this report was prepared by Messrs. W. L. Murphy and W. E. Strohm, Jr., of the Engineering Geology and Rock Mechanics Division (EGRMD), GL, and Mr. G. B. Mitchell, Chief, Engineering Group, Soil Mechanics Division (SMD), GL. Messrs. G. C. Hoff, Chief, Materials and Concrete Analysis Group, Concrete Technology Division (CTD), SL, and P. F. Mlakar of the Structural Mechanics Division (SMD), SL, prepared the structural portion.

COL Nelson P. Conover, CE, was Commander and Director of the WES during the preparation of this report. Mr. F. R. Brown was Technical Director. COL Robert V. Vermillion was District Engineer, Detroit District.

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15 MILE ROAD/EDISON CORRIDOR
SEWER TUNNEL FAILURE STUDY
DETROIT AREA, MICHIGAN

PART I: INTRODUCTION

Study Authority

1. On 10 March 1980 Region V, U. S. Environmental Protection Agency (USEPA), requested that the U. S. Army Engineer Division, North Central (NCD), initiate a technical evaluation (hereinafter designated a study) to determine the cause of a sewer tunnel failure at 15 Mile Road/Edison Corridor, Sterling Heights, Michigan.

2. On 11 March 1980 Region V, USEPA, sent a letter to the U. S. Army Engineer District, Detroit, indicating that under the terms of Appendix A of Interagency Agreement No. EPA-78-D-X0267 the Corps was being assigned a special on-site inspection/study of the sewer tunnel failure.

3. On 12 March 1980 the NCD officially directed the Detroit District to implement the necessary steps required to perform a study of the factors that resulted in the tunnel failure.

4. In a hearing held on 12 March 1980 the Detroit District Engineer, Colonel Robert V. Vermillion, received an official request from the Honorable John Feikens, Chief Judge, U. S. District Court for the Eastern District of Michigan, to investigate the cause(s) of the tunnel failure. However, the Detroit District advised the Court that it had already been asked to perform a study of the failure for USEPA. Judge Feikens agreed that the Detroit District would conduct this study under the aforementioned interagency agreement.

Purpose

5. The purpose of this study was to determine the factor or

combination of factors that caused the sewer tunnel to experience distress.

Scope

6. By direction of the Court, this report is limited to findings of fact and the application of scientific and engineering principles to those facts to draw technical conclusions. This study was limited to a segment of sewer tunnel located in Sterling Heights, Michigan, between 15 Mile Road and the Red Run Drain (approximately 2600 ft), where three failed sections occurred. This segment was a part of the City of Detroit's Contract No. PCI-7, constructed between 10 February 1970 and 15 July 1972. This study reviewed and analyzed the following:

a. Tunnel construction procedures and materials.

b. Geotechnical and groundwater conditions.

c. Structural soundness and design of primary and secondary tunnel linings.

Participants and Coordination

7. In response to the requests from EPA (Region V), Judge Feikens, and NCD, the Detroit District determined the resources, both in-house and Corps-wide, that would be needed to accomplish the tunnel failure study.

8. The failure study was designated as one of the District's top priorities. The District EPA Coordinator was assigned the task of initiating, directing, and coordinating the tunnel failure study. Some of the District's technical resources were made available to assist in the formulation of the entire study. Construction-Operations Division provided a project manager, an assistant project manager, and an on-site resident monitor. The project manager was responsible for coordinating all personnel and activities involved in the study, and acted as the central contact for the District. The assistant project manager assisted the project manager and performed major activities, such as

data collection, safety programming, and procurement processing. The on-site resident monitor observed the bypass (Contract No. CS-858) and temporary repair construction and assisted in the management and coordination of the Corps' field investigation. Construction Branch reviewed the contract and construction records to identify factors or construction difficulties that may have been related to the tunnel failure. The Engineering Division provided geotechnical and design engineers to assist in the review of the tunnel design assumptions and the actual tunnel construction utilized in Contract No. PCI-7.

9. Personnel of the U. S. Army Engineer Waterways Experiment Station (WES) in Vicksburg, Mississippi, were contacted to supplement the resources required to perform the study.

10. On 20 March 1980 the WES agreed to provide a team of professional experts, as required, to evaluate the geotechnical, structural, and construction factors that may have caused the tunnel failure. The scope of work performed by the WES personnel included:

a. Visits to site, as necessary.

b. Assistance in the identification, collection, and review of existing data essential to this study.

c. Determination of additional data requirements (soil borings, concrete cores, etc.).

d. Assistance in preparation of plan of study.

e. Responsibility for testing and analyzing all data collected in the field investigation.

f. Providing written evaluation on sewer tunnel design and construction.

g. Providing written findings of facts on the causes of the tunnel failure.

PART II: PROBLEM IDENTIFICATION

Background

11. As mentioned previously, the segment of sewer tunnel where the failures occurred was part of Contract No. PCI-7. This contract consisted of the construction of approximately 13,633 ft of sewer tunnel. Contract No. PCI-7 was a part of a tunnel network constructed by the City of Detroit to serve Macomb and Oakland Counties. The network, which is several miles in length, had similar designs throughout and was built with the assistance of USEPA grant funds. The site location is shown in Figure 1, General Area Map.

12. The design for this sewer tunnel was prepared by the City of Detroit and is discussed in more detail later in this report. The design assumed that the tunnel would be uniformly loaded externally and be in perfect ring compression. The surrounding soil was assumed to have no shear strength. The concrete wall thickness was a function of the soil cover to the springline and the inside diameter of the tunnel. The compressive strength of the concrete, using these design assumptions, had a safety factor of about 7. For this tunnel, the inside diameter was specified to be 12 ft 9 in. and the wall thickness 16 in. The required concrete compressive strength, f_c, was computed to be 482 psi.

13. The contractor constructed this sewer by boring a circular tunnel with a mining machine and holding the soil in place with steel ribs and wood lagging (see Figure 2, Tunnel Cross Section). Steel forms were centered in the ribs and lagging of the primary lining, and concrete was dropped from the ground surface through baffled pipes to place the concrete lining. Generally, the concrete placements for the lining were 105 ft long. Keyways were formed at the construction joint for each concrete placement. The tunnel design did not provide for any steel reinforcing; however, when the concrete wall thickness was between 15 in. and 12 in., reinforcing was required. Thicknesses less than 12 in. were not allowed and remining was required to provide minimum wall thickness. The contractor sank deep wells to dewater the project

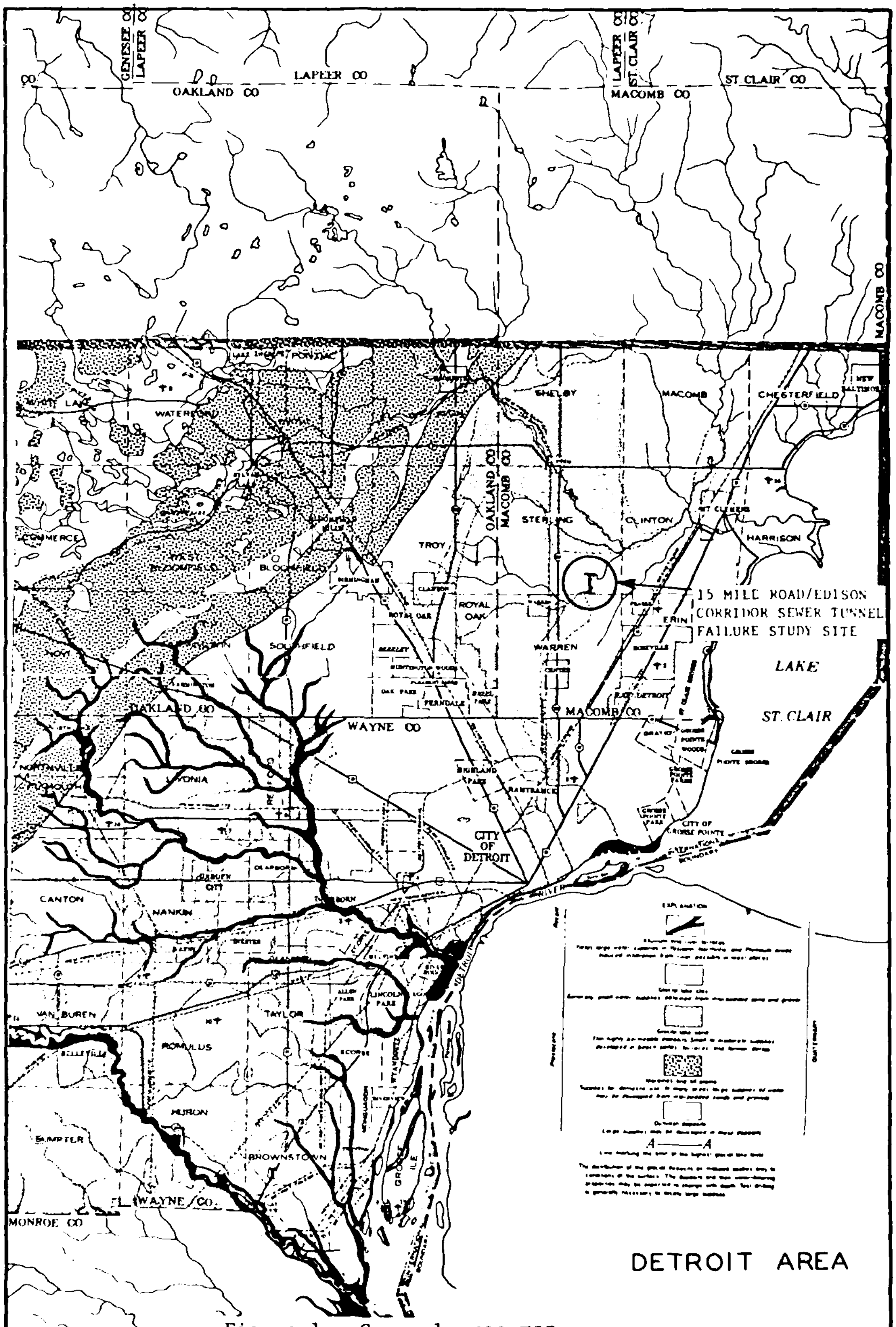


Figure 1. General area map

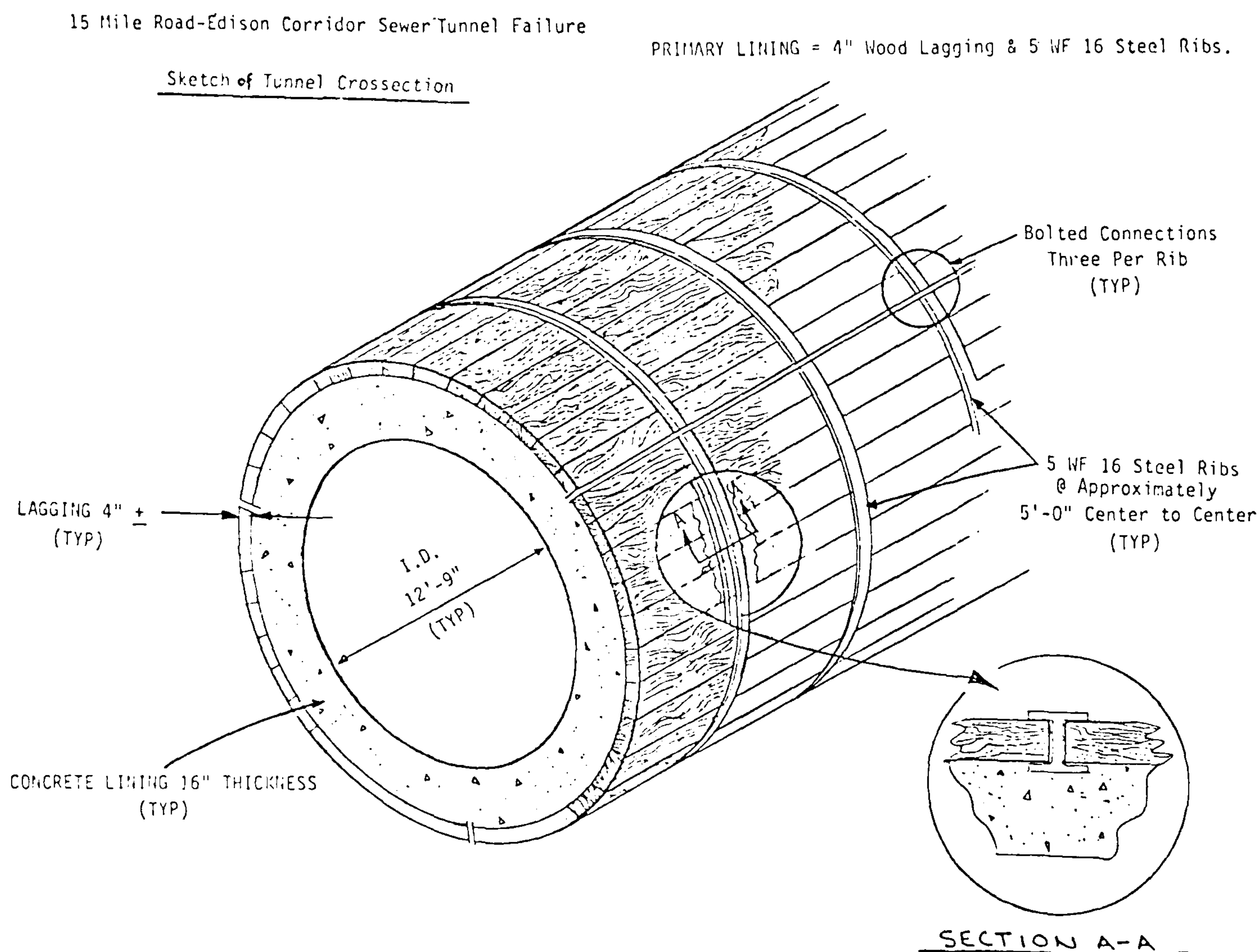


Figure 2. Tunnel cross section

area during construction. The initial specifications did not require the use of waterstops at construction joints, but during construction, the contractor received change orders to add waterstops. Waterstops are either metal (copper or stainless steel) or polymeric (rubber or vinyl chloride) appurtenances which are cast in the concrete of two different adjoining placements in such a manner that they span the joint between the two, preventing the passage of water or other liquids through that joint. The waterstop locations were determined ir the field, based on soil conditions encountered during mining, and are given later in this report.

14. On 12 July 1972 the City inspected the substantially complete sewer tunnel and it appeared to be in an acceptable condition.

15. On 27 January 1980 the Detroit Water and Sewerage Department (DWSD) sewer safety crew performed an inspection of Contract PCI-7. The crew entered the tunnel through a manhole just south of 15 Mile Road, walked one mile southward, and exited from a manhole south of the center line of 14 Mile Road. It was during this inspection that the three distressed areas in the sewer tunnel were discovered (see Figure 3, Tunnel Plan and Elevation Sketch). No record of an inspection between 1972 and 1980 was found.

Study Chronology

16. On 8 February 1980 the Detroit District initiated an on-site monitoring assignment at the bypass construction site (Contract No. CS-858) located in the Edison Corridor between 15 Mile Road and the Red Run Drain, Sterling Heights, Michigan.

17. On 18 March 1980 Detroit District personnel were in the DWSD offices collecting contract documentation and history concerning Contracts No. PCI-7 and No. CS-858. On 30 May 1980 a letter was transmitted to the DWSD requesting clarification on some aspects of the contract documentation and history reviewed. On 20 June 1980 the DWSD provided a written response to our inquiries. The following contract documentation and historical items were reviewed and are on file

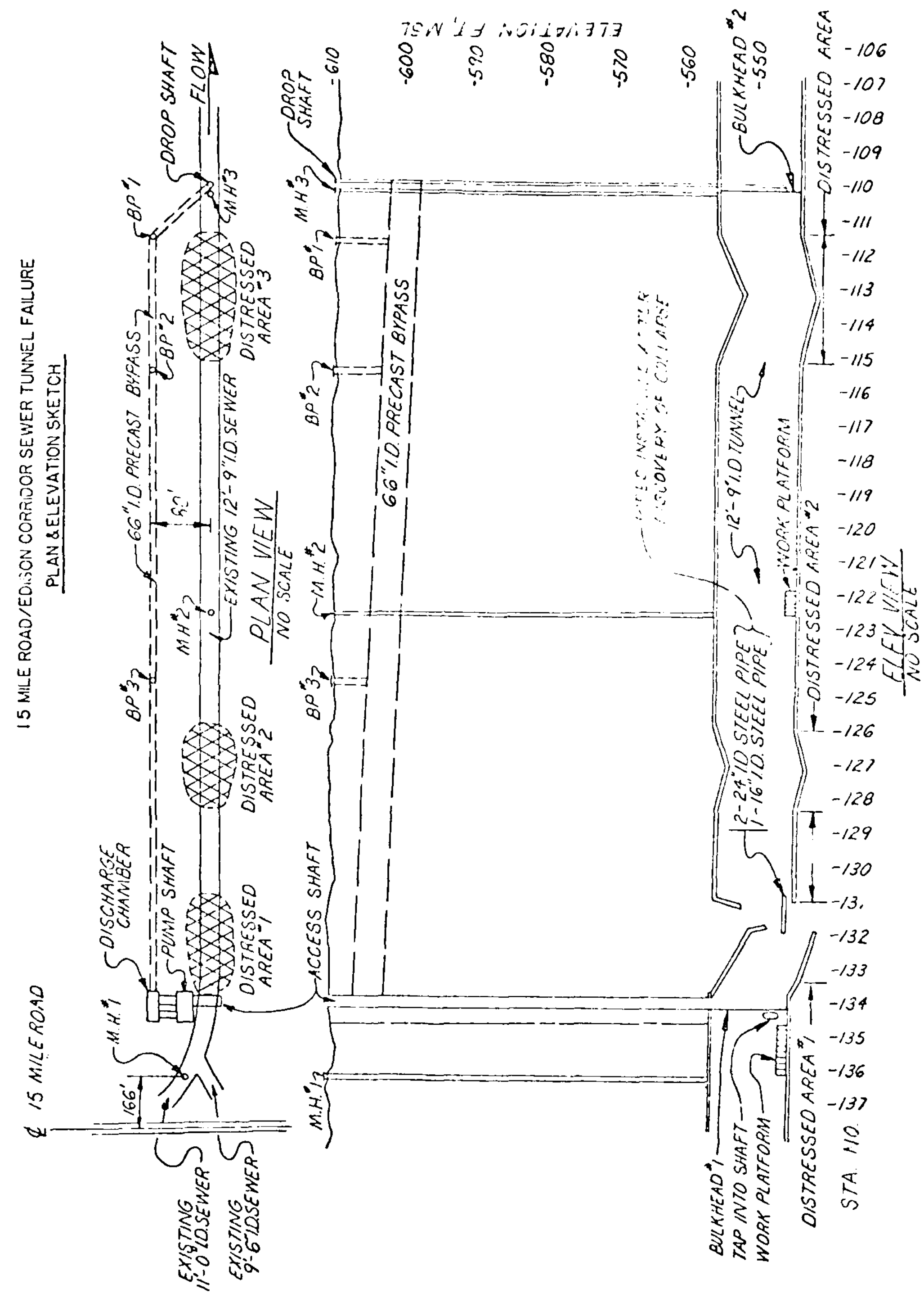


Figure 3. Tunnel plan and elevation sketch

at the Detroit District and the WES:

- a. A set of Contract No. PCI-7 as-built plans and specifications.
- b. The plans for Contract No. CS-858 (no specifications issued for this construction; work under emergency status).
- c. Inspector's reports and daily reports for PCI-7.
- d. Soil data and boring logs for PCI-7 and CS-858.
- e. Design calculations for Contract No. PCI-7.
- f. Shop drawings for PCI-7.
- g. Groundwater data collected during Contract No. CS-858.
- h. Tunnel profiles proposed under CS-858.

18. The City made a videotape of the distressed areas and the Detroit District made complete pictorial surveys of the tunnel (Contract No. PCI-7, Sta 136+00 to Sta 110+00) which are on file at the Detroit District and the WES.

19. On 21 April 1980 a WES drilling crew and equipment arrived at the bypass construction site. All but two of the soil borings were completed and the samples delivered to the WES for analysis and testing on 21 May 1980. Samples from the two remaining soil borings and all the concrete cores were delivered to the WES on 25 June and 9 July 1980. A large piece of concrete was taken out of Distressed Area No. 1 and delivered to the WES on 7 August 1980.

20. During the period May-August 1980, the WES performed laboratory tests on the soil and concrete samples obtained from the site.

21. On 15 June 1980 a plan of study was presented to the USEPA (Region V) and NCD. This plan of study presented the analysis approach by which the factors contributing to the tunnel failure would be determined.

PART III: STUDY METHODOLOGY

Construction

Chronology

22. On 16 December 1969 the Common Council for the City of Detroit approved the award of Contract No. PCI-7 to the low bidder, Michigan Sewer Construction Company, in the amount of $8,977,765. This contract consisted of the construction of approximately 13,633 ft of sewer tunnel. The north and south limits of this contract were designated as Sta 136+33 and Sta 0+00, respectively. (This study was restricted to that segment of tunnel between Sta 136+02 and Sta 103+00 in which three distressed areas occurred.)

23. On 10 February 1970 the City issued a start work order to the contractor. The contractor started this construction project by constructing a work shaft at Sta 51+66. The contractor intended to mine north and south from this shaft. However, mining in the north heading was halted on 1 September 1971 when water and mud from the Red Run Drain flooded the tunnel at approximately Sta 103+23.

24. On 13 October 1971 the contractor placed a "Caldwell" mining machine in the work shaft at Sta 136+02. The contractor started mining this new south heading on 27 October 1971.

25. Between 27 October 1971 and 24 May 1972, the contractor mined the new south heading between Sta 135+86 and Sta 103+63. See Table 1, Excavation Summary, for a detailed breakdown of starts and stops of mining, feet mined per day, number of steel ribs between stations, soil conditions encountered, problems encountered, and significant events such as remining and resetting of ribs. These data were obtained by a thorough search of all available records.

26. Between 13 March 1972 and 21 June 1972, the contractor placed the concrete lining between Sta 135+82 and Sta 103+47. See Table 2, Concrete Summary, for a detailed breakdown of the concrete placement dates, stationings, durations, operations, problems, and notes such as waterstop locations and resteel installation. These data

were also obtained during the records search.

Plans and Specifications/Change Orders

27. The plans and specifications for Contract No. PCI-7 were collected and reviewed for this study; however, they were not included in the body of this report. These plans and specifications will be kept on file in both the Detroit District Office and the WES for review by interested parties.

28. Some changes were made to the contract plans and specifications. The documented contract change orders are as follows:

a. Change Order No. 1 extended the contract substantial completion date of 1 July 1971 to 21 July 1971, and the final completion date of 30 September 1971 to 20 October 1971. This time extension was precipitated by the occurrence of an operating engineers strike (Local 324) from 1 September 1970 to 20 September 1970. The contractor requested a 26-day time extension but the City only allowed him a 20-day extension. The total contract price was not affected by this change order. The issuance of the change order was approved by the Board of Water Commissioners on 26 July 1971.

b. Change Order No. 2 provided for the installation of waterstops in certain portions of the tunnel where unacceptable soil conditions were encountered. The waterstops were intended to prevent imminent infiltration of water and soil into the tunnel and thereby eliminate potential damage due to loss of ground support (see Table 2, Concrete Summary, for waterstop locations). The contractor was to install waterstops, where required, on a cost plus-limited amount basis. The cost increase for this change was not to exceed \$3400 (\$2000 for split bulkhead and \$1400 for the waterstops; approximately 7 waterstops were anticipated). No time extension was allowed for this change. The issuance of this change order was approved by the Board of Water Commissioners on 10 January 1972 and by the City's Common Council on 18 January 1972.

c. Change Order No. 3 provided for the construction of one interceptor bulkhead south of the Red Run Drain. This change deleted a bulkhead from Contract No. PCI-6 and allowed the construction of an exit shaft between these two adjoining contracts. Accordingly, a deduction of \$1000 was approved for Contract No. PCI-6. The bulkhead was relocated into Contract No. PCI-7. No time extension was allowed for this change order. The issuance of this change order was approved by the Board of Water

Commissioners on 20 March 1972 and by the City's Common Council on 28 March 1972.

d. Change Order No. 4 provided for the installation of waterstops in all remaining concrete construction joints and the alteration of three steel bulkheads to accommodate the waterstops. This change order increased the contract price by an amount not to exceed $9600, with the final price to be fixed by a confirming change order when the cost was determined ($6000 for the three bulkheads and $3600 for the waterstops; approximately 18 waterstops were anticipated). No time extension was allowed under this change order. The issuance of this change order was approved by the Board of Water Commissioners on 30 May 1972 and by the City's Common Council on 13 June 1972.

e. Change Order No. 5 established the final price for Change Orders Nos. 2 and 4. The work under the initial two change orders (Nos. 2 and 4) had been completed and the total amount owed the contractor was determined to be $11,078.18. The work under this change order included alteration of 4 split bulkheads and installation of 19 waterstops. No time extension was allowed under this change order. The issuance of this change order was approved by the Board of Water Commissioners on 29 January 1973.

f. Change Order No. 6 deleted the required bulkhead on Contract PCI-7 at the junction between Contract Nos. PCI-7 and PCI-12A. The contractor requested deletion of the bulkhead because of coordination problems with adjoining work performed by others. The City agreed that it would be more advantageous to have the bulkhead constructed under Contract No. PCI-12A. This change order reduced the contract price by $1500. This change order had no effect on the contract completion dates.

29, Certain deviations to the contract plans and specifications occurred during the construction of this sewer tunnel. These deviations are as follows:

a. Detailed Specifications, Section 1.III.3, Tunnel Excavation, Paragraph C, Full Circle Steel Ribs, page 1.7, required that no part of ribs will, when in place, protrude into the specified tunnel wall thickness and that rib sections shall fit closely for bolted connections at segmental and transverse joints. Also, the bolt connections were to be capable of developing the full strength of the ribs. At least some of the ribs which were installed, as documented by the during-construction photographs, and ribs removed at the access shaft during the

bypass construction had bearing plates that extended past the interior flange of the ribs and into the tunnel wall thickness. The full circle ribs were oriented so that bearing plates were located at the invert, at the 10 o'clock and the 2 o'clock positions. As the mining machine advanced, the steel ribs were hydraulically jacked out against the surrounding soil. This jacking left a space between the bearing plates at the 10 and 2 o'clock positions. Steel spacers were placed between the bearing plates. The approved shop drawing shows the rib bearing plates with bolt holes located on both sides of the rib web. However, construction photographs show that the bolt holes were actually located on that portion of the bearing plate which extended past the interior flange of the ribs. During-construction photographs confirm the above-mentioned condition.

b. Under this same section of specifications, a requirement was added by Bulletin No. 5 where in no case shall the soil be supported only by ribs and wood lagging for a period longer than seven consecutive days. The Inspector's Reports indicate that there was approximately a 5-month delay before the concrete lining was placed inside the full circle rib and wood lagging. Mr. Beckham's 2[illegible] June 1980 response to our 30 May letter of inquiry confirmed that this time lag was approved by the City.

c. Detailed Specifications, Section 2.II.3, Fine Aggregate (Sand), Paragraph A, Physical Requirements, page 2.2, required testing for organic impurities, tensile strength, and soundness. The test reports for these physical requirements were not found during our review of the contract documentation and files.

d. Detailed Specifications, Section 2.II.4, Coarse Aggregate, Paragraph A, Physical Requirements, page 2.4, required testing for soundness, loss by washing, other deleterious substances, and thin and elongated pieces. The test reports for these physical requirements were not found during our review of the contract documentation and files.

e. Detailed Specifications, Section 2.II.4, Coarse Aggregates, Paragraph B, Gradation, page 2.5, required that only 60 to 90 percent of the 6A coarse aggregate pass the 1-in.-sq sieve openings. All the Metro Water Department-Detroit Division of Engineering Mechanical Analysis Reports showed that 100 percent of the 6A coarse aggregate passed the 1-in.-sq sieve openings.

f. Detailed Specifications, Section 2.III.2, Forms, Paragraph E, Removal of Forms, page 2.10, specified that the

time of removal of forms shall be at least 36 hours after concrete placement was terminated. Discussions with the City of Detroit's inspector on this construction contract disclosed that the majority of the south heading utilized 140 ft of tunnel forms. A review of the City's records indicates that concrete was generally placed in 105-ft segments. The practice was to strip the remaining 35 ft of the second to the last placement and the upstream 70 ft of the last placement to form the next 105 ft. This left 35 ft of forming in place on the downstream end of the last placement. When concrete was placed every day, the day shift would strip forms, reset forms, and place concrete. The second shift cleaned up ahead of forms, removed form bolts, and began stripping forms (no third shift). In the area of this study, the duration between concrete placements was approximately 16 hours. This means that the last placement had 70 ft of forms stripped within 16 hours after placement, less than one-half the specified minimum.

g. Detailed Specifications, Section 2.III.6, Mixing Concrete, Paragraph F, Samples for Concrete Strength, page 2.16, specified that three standard test cylinder specimens be made for each run of 50 cu yd or less, and for each 100 cu yd poured. The available contract records on concrete sampling show that the average concrete pour in the study area was approximately 260 cu yd for which only two cylinders were normally made.

h. Detailed Specifications, Section 2.III.7, Placing Concrete, Paragraph B, Concrete Drop Holes, page 2.17, required that the drop pipe have a pocket or remixing box at the lower end to permit remixing of the concrete before it flowed into the forms. Also, this paragraph specified that a flexible pipe or boot be connected to the side of the remixing box on the drop pipe (extending around the arch of the forms and down to the springline) to reduce the free fall of the concrete. Mr. Beckham's 20 June 1980 response letter states that the drop pipe end consisted of a 90-deg elbow and no mention was made of the pocket for remixing nor the use of a flexible pipe or boot. A discussion with the City's inspector confirmed that the concrete was placed through a drop pipe without using the specified pocket and flexible pipe or boot.

i. Detailed Specifications, Section 2.III.8, Construction Joints, Paragraph A, General, page 2.18, specified that the contact surface of any joint would, after being cleaned, be coated with a waterproof material before adjoining concrete was placed. In particular, the vertical construction joints would be coated with a cold

bituminous, fibrous compound Flinkote #200 or equal. Inspections made during our field investigation and cores taken at joints revealed no mastic coating.

j. Detailed Specifications, Section 2.IV.1, Core Tests - Tunnel Sewers, page 2.22, required that three 6-in.-diam cores be taken for each 1000 ft of tunnel sewer or fraction thereof. Also, these cores were to be measured for length and tested for compressive strength. Mr. Beckham's 20 June 1980 response letter verified that the required number of cores were not taken and, of the cores taken, no test results or records of visual observation were found.

Comments

30. The Construction Branch of the Detroit District has made a comprehensive evaluation of the construction techniques and procedures used on this project. Several pertinent comments were generated during the evaluation. These comments are as follows:

a. A 4- to 5-month time lag between setting ribs and lagging and placement of the concrete lining occurred on this project. Current sewer tunnel construction contracts in the DWSD do not allow such long lag periods.

b. The Portland Cement Association in their "Suggested Specifications for Plain and Reinforced Concrete, 1969" states that "concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to flowing." On this project concrete was lowered to the tunnel from the ground surface through drop holes. In the average placement length of 105 ft, two drop holes were installed. One drop hole was located about 7 ft downstream from the previous pour. The second drop hole was located about 50 ft downstream of the first drop hole. Discussions with the City's inspector revealed that the first drop hole was used for the entire pour unless it became plugged. Therefore, concrete in this project was normally transported to distances up to 100 ft in the forms.

c. The Portland Cement Association in the same reference as above suggests a minimum of two concrete cylinder specimens for each test at each age, and not less than one test for each 150 cu yd of concrete. The Report of ACI Committee 301, "Specifications for Structural Concrete," recommends three specimens for each test and one test be made for each 100 cu yd of concrete. On this project it was common practice to take only two specimens for concrete placements in excess of 250 cu yd.

d. According to the Inspector's Reports, the machine mining was higher than the specified grade and off line to the west by as much as 1.75 ft in Distressed Area No. 1. The high areas were generally remined. During the remining, the ribs and lagging were removed and modified ribs were reinstalled.

e. The hydraulic system of the "Caldwell" mining machine was continually failing.

f. Between 2 February 1972 and 21 June 1972, water seepage was experienced through the primary liner throughout the mined tunnel. On 30 May 1972, mud and water entered the tunnel during mining through a drop hole drilled through a field tile (Sta 118+57). On 21 June 1972 flooding occurred in the tunnel in the area of Sta 103+67.

g. On 15 December 1971 the rear shield of the "Caldwell" mining machine developed a radial crack, and on 8 January 1972 the tail section broke.

h. The Inspector's Reports note that a void occurred over the mining machine from Sta 109+36 to Sta 103+63. Grouting was done over the mining machine at Stas 104+30, 104+25, and 104+10.

i. Overmining with mining machine occurred between Sta 109+36 and Sta 103+83.5.

j. Ribs from Sta 107+40 to Sta 107+65 were deflected by excessive loading and were braced.

k. The Inspector's Reports show that voids occurred between steel forms and rib and lagging during concrete placement at Stas 134+92 and 126+12. Voids were filled at a later date.

l. Cold joints were noted in the Inspector's Reports at Sta 130+17 (unspecified locations), Sta 130+35 (east and west sides), Sta 130+10 (lower east quarter), Sta 123+50 (invert), Sta 123+30 (west side), Sta 118+62 (east and west lower quarters), Sta 114+42 (east and west sides), and Sta 112+32 (east and west sides).

m. The Inspector's Reports state that pour progress was slow for concrete placements from Stas 133+32 to 132+27, 129+12 to 128+07, 127+02 to 125+97, and 119+67 to 118+62. This slow progress was caused by late arrival of mix trucks and plugged drop pipes. The Inspector's Reports regarding the placement starts and terminations indicate that an average rate of 65 cu yd/hr constituted normal progress. Slow progress would appear to be something less than this rate.

n. The Inspector's Reports stated that the concrete between Sta 129+12 and Sta 128+07 was placed "wet" and that

groundwater was entering the forms. No slump test was performed on the concrete. It is assumed that "wet" concrete was a visual evaluation of high slump.

Geotechnical

31. Geotechnical studies included review and analyses of pertinent information from the City and other sources, site visits, surface borings, tunnel wall holes, laboratory tests of soil samples, and analytical studies. The studies were made to develop the geology of the site, groundwater conditions, soil stratification, soil properties and in situ soil conditions, and ground pressures. The main objective was to assess the in situ soil conditions before, during, and after construction, and after failure, and their possible bearing on the cause of tunnel distress. The studies made and pertinent results are summarized in this section.

Investigations performed

32. The initial step was a geotechnical plan of study which was based on a review of a preliminary copy of CS-858 contract drawings for the temporary bypass and pumping shaft containing boring logs for the study site. A viewing of the City of Detroit video tape of tunnel Distressed Areas 1 and 2 provided insight to the problem being studied. This information was used to select the type and location of surface borings and needed laboratory tests. Other information used, field work performed, laboratory tests conducted, and analytical studies performed are outlined below.

33. Available information. Information from the City of Detroit, Detroit District, and other sources used in the geotechnical study is summarized in Table 3. Pertinent literature used in the study is cited where appropriate. The preconstruction boring logs provided general information on soil stratification and physical properties (standard penetration test, blow counts, water contents, wet unit weights, and unconfined shear strengths) and were used in developing

area geology and possible sources of groundwater. The construction records for PCI-7 contained information on the following:

- a. Construction sequence.
- b. Dewatering well locations and depths.
- c. Excavated soils and groundwater seepage during tunneling.
- d. Distances between concrete forms and lagging, recorded by stations at 4 to 9 locations for each pour.
- e. Locations of construction joints, water stops, cold joints, leaks, joints requiring concrete finishing work.
- f. Locations of crown drill holes to check tightness of concrete against lagging.

Contract CS-858 site investigation data provided information on detailed soil stratification, soil conditions, grout quantities, dewatering operations, piezometer levels along the tunnel and approximately 1300 ft east and west of the tunnel, and profiles of the tunnel invert, crown, and springlines.

34. Fieldwork performed. The surface borings, site visits, and disturbed sampling through holes in the tunnel are described below:

- a. Surface borings. WES surface borings, under the supervision of a field geologist, were started on 22 April 1980 to confirm soil stratification and in situ soil conditions along distressed and nondistressed segments of the tunnel. The borings were needed to obtain undisturbed samples for laboratory tests to develop soil properties for analyses and analytical studies. A total of seven sample borings and three wash borings (to define a land fill found in the second boring) were made. The boring locations and purposes are summarized in Table 4. A boring location within Distressed Area 1 was not possible because of ground freezing and shaft excavation operations. A 24-in. gas line and overhead power lines west of the tunnel restricted boring locations to the east side (except for boring WES-10). Overhead power lines and towers further restricted locations for borings on the east side. Boring logs are contained in Appendix A. All borings were made using a Failing 1500 truck-mounted drill rig, as shown in Figure 4a, drilling mud, and casing where necessary. Borings WES-1 through WES-4 were drilled deep enough to penetrate a thick sand stratum encountered near the tunnel invert. The borings bottomed in hard sandy clay to clayey sand,

Figure [illegible]. Field Investigation Methods, 15 Mile Road/Edison Corridor Tunnel Failure Study

identified as glacial till, underlying and marking the bottom of the sand stratum. Boring WES-5 bottomed about midway through the sand stratum. Borings WES-6, WES-7, and WES-8 were drilled only deep enough to define the landfill (maximum depth 29 ft). Logs were not made of these borings. All boreholes were grouted when completed.

(1) Sampling. Undisturbed samples were taken using a Hvorslev fixed piston sampler with thin-wall steel tubes where possible. Three-inch or five-inch diameter tubes, with spring steel catchers riveted in the bottom, were pushed when possible in sandy soils to prevent the sample from sliding out of the tube during withdrawal from the borehole. Soils too hard to push the Hvorslev sampler into were sampled by rotating a Denison barrel equipped with a 5-in. diam Hvorslev inner tube and a 6-in. carbaloy rock bit. Cohesive samples were pushed out of those 5-in. steel tubes not fitted with catchers and waxed in 6-in. diam cardboard tubes. Samples containing sand were left in the sample tube. Jar samples were taken for each undisturbed sample. Disturbed samples were taken using a standard (2-in. O. D.) split spoon with catcher. Disturbed samples were sealed in glass jars. All samples were delivered to the WES soils laboratory by a Detroit District van on 7 June 1980.

(2) Penetration tests. Penetration resistance tests were made in borings WES-1, WES-2, WES-9 and WES-10 using the standard 2-in. O. D. split spoon and a 140-lb weight free falling 30 in. Blow counts were recorded for each 6-in. penetration up to 18 in. In dense sands, a penetration of 18-in. was not feasible and the blow count for the first 6 in. plus the blow count in excess of about 50 to 80 for an additional penetration less than 6 in. were added together and divided by the total penetration in tenths of a foot to obtain an equivalent blow count for a 1-ft penetration.

b. Site visits. Visits were made to the site by WES geotechnical engineers on 7-8 May 1980 and 22-26 June 1980. The first visit was made to check boring work and sampling problems (recovering sand samples), to observe accessible tunnel conditions north of Distressed Area 1 at the work shaft, and to obtain additional available records and information from site personnel. The second visit was made to observe completion of concrete coring as shown in Figure 4b, check soil conditions being found behind the lagging from 2-in.-diam holes drilled through the tunnel wall, supervise soundings using a small cone penetrometer in holes south of Sta 130+25, and then

inspect conditions at seeps, joints, and cracks along the tunnel south of Distressed Area 1.

c. Tunnel wall holes. A total of sixty-four, 2-in.-diam holes were drilled through the tunnel wall at the invert, lower quarter, and springline to check soil conditions behind the lagging in both distressed and nondistressed areas. Voids or soft loose soil conditions and types of soils encountered were of primary interest. The holes were drilled through the concrete using an air hammer as shown in Figure 4c and through the lagging using a wood auger or circular saw bit. The holes were not always perpendicular to the surface and concrete thickness and measurements taken were not true indications of wall thickness. The soil behind the lagging was sampled using a common piece of 1-in. pipe, 4 ft long, and small sledge hammer. Soil recovered was classified and placed in plastic or glass jars. A small hand-held 60 deg cone penetrometer (0.5 in^2 area) was used before sampling in most of the holes south of Sta 130+25 to obtain a qualitative comparison of in situ soil consistency (loose or dense for sands and soft or stiff for clays).

35. Laboratory test program. Laboratory tests performed on selected undisturbed samples included classification and physical property tests (natural density and water content, specific gravity, and gradation). Consolidation tests were performed on two clay samples, one from a depth of 21.5 ft and one from a depth of 55.3 ft, to determine the degree of overconsolidation of upper and lower clay stratum. Five drained direct shear tests were performed on clay samples from depths of 10, 25, 41, 50, and 52 ft to determine drained shear strength. Special consolidated undrained triaxial compression tests with pore pressure measurements ($K_o \bar{R}$) were performed on four undisturbed samples from depths of 55 ft to 65 ft and included clays and silty sands to determine modulus values and estimates of in situ K_o values (K_o = coefficient of earth pressure at rest = ratio of effective horizontal earth pressure to effective vertical earth pressure) for analytical studies. Special tests were also performed on undisturbed samples to assess the piping characteristics (susceptibility to erosion through small holes or cracks under various groundwater levels) of fine-grained sands, silts, and sandy clays. The

special $K_o \bar{R}$ tests and piping test procedures are outlined briefly below. Other tests were conducted using standard procedures described in Engineer Manual EM 1110-2-1906.

a. K_o R tests. Specimens 2.8 in. in diameter and 6 in. high were trimmed in the humid room from selected samples. The specimens were enclosed in a rubber membrane, placed in a triaxial chamber, and saturated under isotropic back pressure and then consolidated to the estimated in situ vertical pressure. A girth clamp placed around the specimen to monitor lateral deformations was used to adjust lateral pressures during consolidation such that no change occurred in the diameter of the sample, (i.e., no lateral strain to simulate in situ K_o conditions). The ratio of lateral pressure to vertical pressure at the end of consolidation provided an estimate of the in situ K_o value. The slope of the stress-strain curve during axial loading provided an estimate of in situ modulus which for a linear elastic material would be Young's Modulus. The test also provided an estimate of in situ shear strength.

b. Special piping tests. A special apparatus shown in Figure 5 was constructed to determine piping characteristics of the undisturbed samples of sands and silts. The following procedure was used on performing the special soil piping test:

(1) Measured desired length of sample for testing and cut off excess part of sample tube at each end.

(2) Cut narrow slot in sample tube, longitudinally, along center one-third of sample length or short circumferential slots around middle of sample.

(3) Measured total weight of cut sample and tube.

(4) Attached end caps to sample tube with a 200-mesh screen placed inside each end cap.

(5) Placed rubber sleeves over end caps and the sample tube.

(6) Placed band clamps over rubber sleeves, making end caps watertight.

(7) Hooked each end cap to pressurized water source.

(8) Placed sample on a cradle inside plexiglas reservoir and filled reservoir with water to submerge sample to increase percent saturation. Sealed slots with tape and applied water at 1/2-psi pressure to ends of sample to further increase percent of saturation.

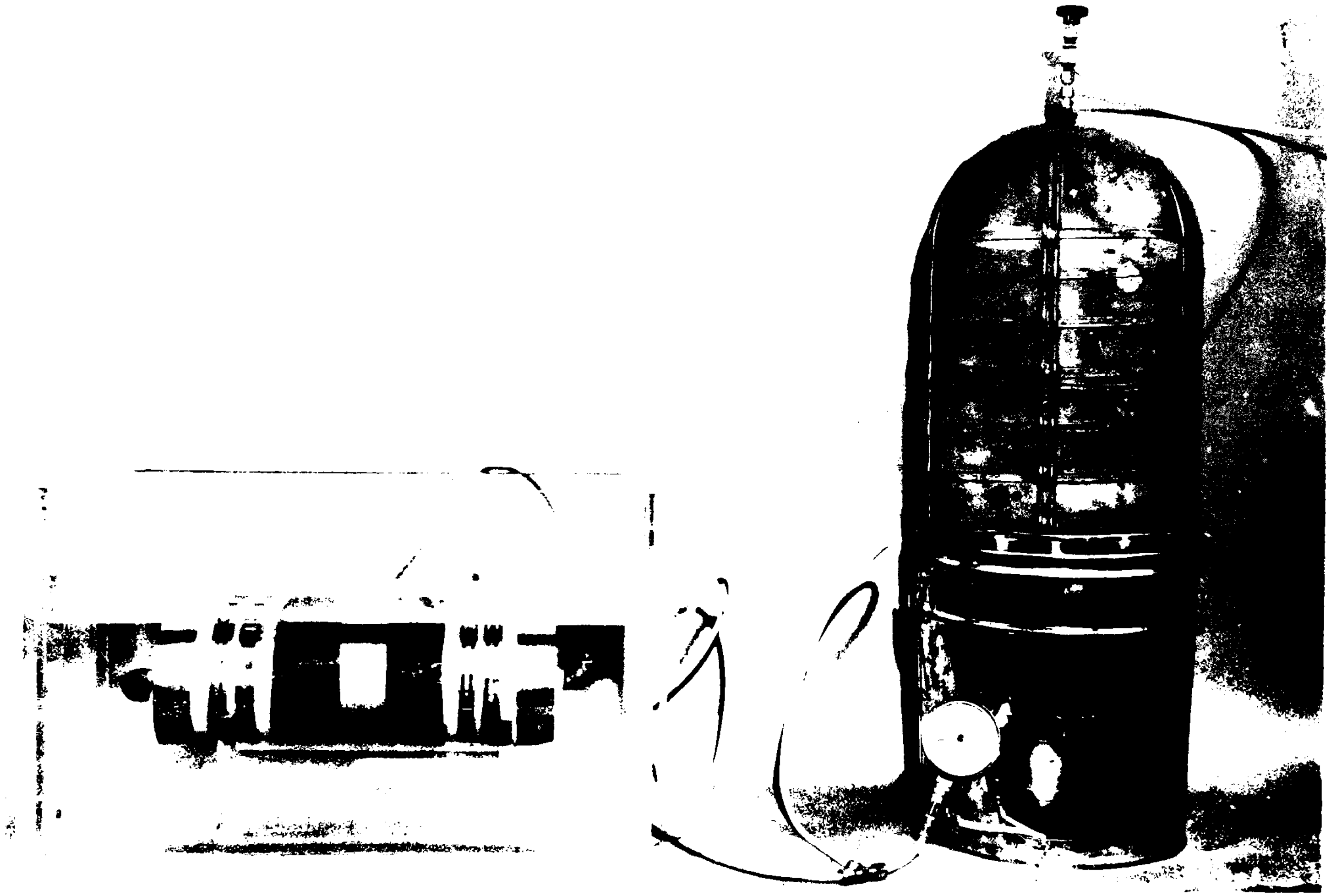


Figure 5. Special test apparatus for measuring piping characteristics of sands and silts
15 Mile Road/Edison Corridor Tunnel Failure Study

(9) After increasing percent saturation, opened slots, applied water under increasing pressure to initiate piping, and recorded water pressure and time.

(10) Applied increasing water pressure until all piping ceased as indicated by no further flow of material out of slots.

(11) Dismantled sample and noted erosion conditions at ends of sample.

(12) Measured weight and gradation of material piped out of sample and material left in the sample.

(13) Measured weight of empty sample tube.

The test apparatus in Figure 5 shows a 5-in.-diam sample in its steel sample tube being saturated under 0.5-psi water pressure (circumferential slots cut around the center of the steel tube are sealed with gray tape).

c. Pinhole tests. Pinhole erosion tests described in EM 1110-2-1906, Change 1, dated 1 May 1980, were performed on three samples of a sandy clay to determine the susceptibility of this material to seepage erosion through small openings. The test consists of measuring the rate of water flow through a 1/6-in. diam hole in a 4.6-in. long sample under a range of water pressures. The color of water (cloudy, clear, etc.) flowing out of the hole is recorded. The flow rate versus water head is used to assess erosion and to determine a dispersion index for the soil.

36. Analytical studies. In addition to area and site geology and groundwater studies, analyses were made to determine earth pressure coefficients, in situ stresses, and elastic constants (Young's Modulus and Poisson's ratio) of the soils. Changes in the in situ stresses after construction and potential water and soil infiltration into the tunnel also were analyzed. The results were provided to WES structures engineers for use in soil-structure interaction analyses (paras 94 and 95). Other possible causes of distress such as seimic ground vibrations (para 55) were considered.

Regional geology

37. A brief description of the Quaternary stratigraphy of southeastern Michigan is included in this report to enable a better understanding of geo' .: strata and soils encountered locally at the study site. The ve l geologic col umn in southeastern Michigan consists typically ot -400 ft of relatively flat-lying glacial soils, deposited within the last one million years, overlying dipping bedrock strata of early Paleozoic age (300 to 500 million years old). Bedrock is below the depth of interest for this geotechnical investigation and will not be discussed further, except to say that occurrences of methane (natural gas) in the glacial overburden probably originates from the underlying bedrock from which the gas is believed to migrate upward (Mozola, 1973).

38. Glacial geology. During the latter part of the Wisconsin Glaciation, the western half of southeastern Michigan was covered by a large glacial lake. The level of the lake was controlled by the glacier as it retreated to the east and north (Twenter, 1975). Beaches were formed when the lake level remained stationary for some time. Clays and silt were deposited toward the center and deeper parts of the lake. As the glacier advanced or retreated, the beach deposits would be reworked and deposited higher or lower than previous beaches forming offlap and onlap cycles of deposition with facies changing from sand to clay. Morainal deposits were moved about and redeposited by the lake water. Figure 6 is a map showing the location and type of glacial deposits.

39. Glacial deposits. In southeastern Michigan, glacial deposits are variable, ranging from soft and hard clays to silts to sands and gravels. The strata can persist for miles laterally or play out within tens of feet by lensing or by gradational facies changes. The glacial section is highly variable and complex both laterally and vertically. All glacially deposited materials are known as drift and were transported and deposited or reworked by ice, water in streams, water in lakes, or by wind. Unstratified and unsorted drift was deposited by ice and is a mixture of clay, boulders, sand,

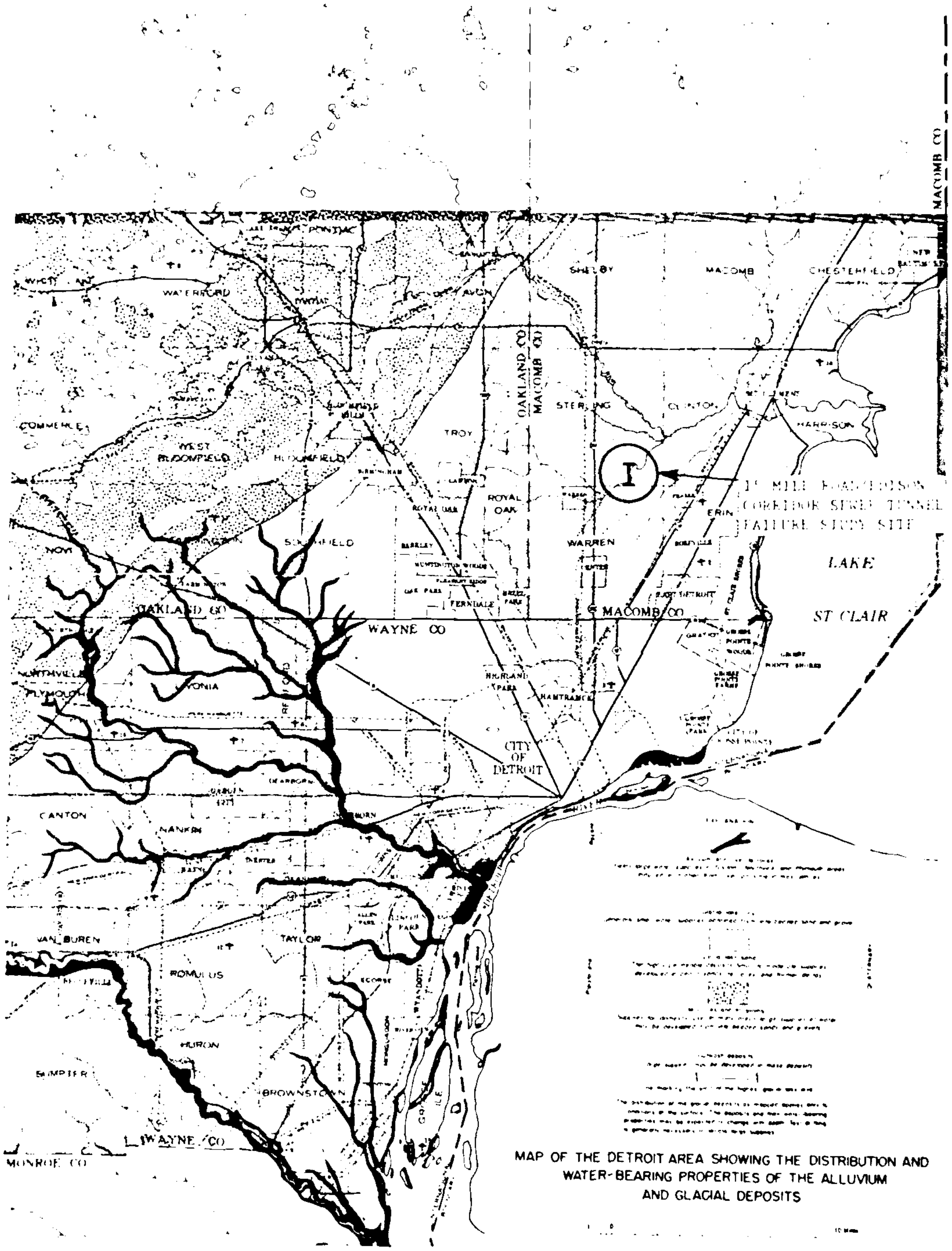


Figure 6. Location and type of glacial deposits, 15 Mile Road/Edison Corridor Tunnel Failure Study (Wisler, et al., 1952)

and silt known as till. Till is commonly hard when drilled and often is described locally as "hardpan" in drillers' logs. Stratified drift is deposited by and within water of streams and lakes or by wind and includes outwash deposits and lake deposits. Outwash deposits are generally sands and gravels carried beyond the ice by meltwater streams and are recognized in subsurface investigations as channel shaped strata that form lenses of limited lateral extent in cross sections. Lake deposits consist of water-laid till, bottom sediments, ice-rafted (berg) sediments, and near shore sediments deposited as beach ridges and deltas (Flint, 1971). Bottom sediments are commonly the finer grained constituents, clay and silt; beach ridges and deltas are usually sand and gravel. Fine-grained lake sediments are often laminated in cycles representing yearly seasonal deposition of clays and silts and are often varved. Environments of deposition sometimes overlap both laterally and vertically (time wise), producing complex and intermingled depositional sequences.

40. Loading from glacial ice. The last glacial ice which covered the St. Clair basin retreated north approximately 14,000 years ago (during the Early Post-Lake Border interval of the Cary substage of the Wisconsin stage of glaciation) (Hough, 1958). Lacustrine clays were deposited in the Lake St. Clair basin during the period of glacial Lakes Whittlesey and Warren, approximately 13,000 to 11,000 years ago (Soderman and Kim, 1970). Paleontological studies indicate that a dry climate existed in the Great Lakes region from 11,000 to 9,000 years ago (Stanley-Chippewa interval of Hough) (Dreimanis, 1970). The effects of the dry climate and associated isostatic changes in the Great Lakes Region caused a lowering of the groundwater table due to a lowering of the Lake St. Clair level. Based on the elevation of overconsolidated brown oxidized zones in the clay, the groundwater table at the time of this dessication and oxidation was at least 12 to 13 ft lower than it is now in the Detroit and Windsor, Ontario area (Dreimanis, 1970). The lowering of the groundwater table caused the development of the oxidized overconsolidated crust. Based on the

above geologic history, it does not appear that the lake bed clays and beach sands have been subjected to ice loading.

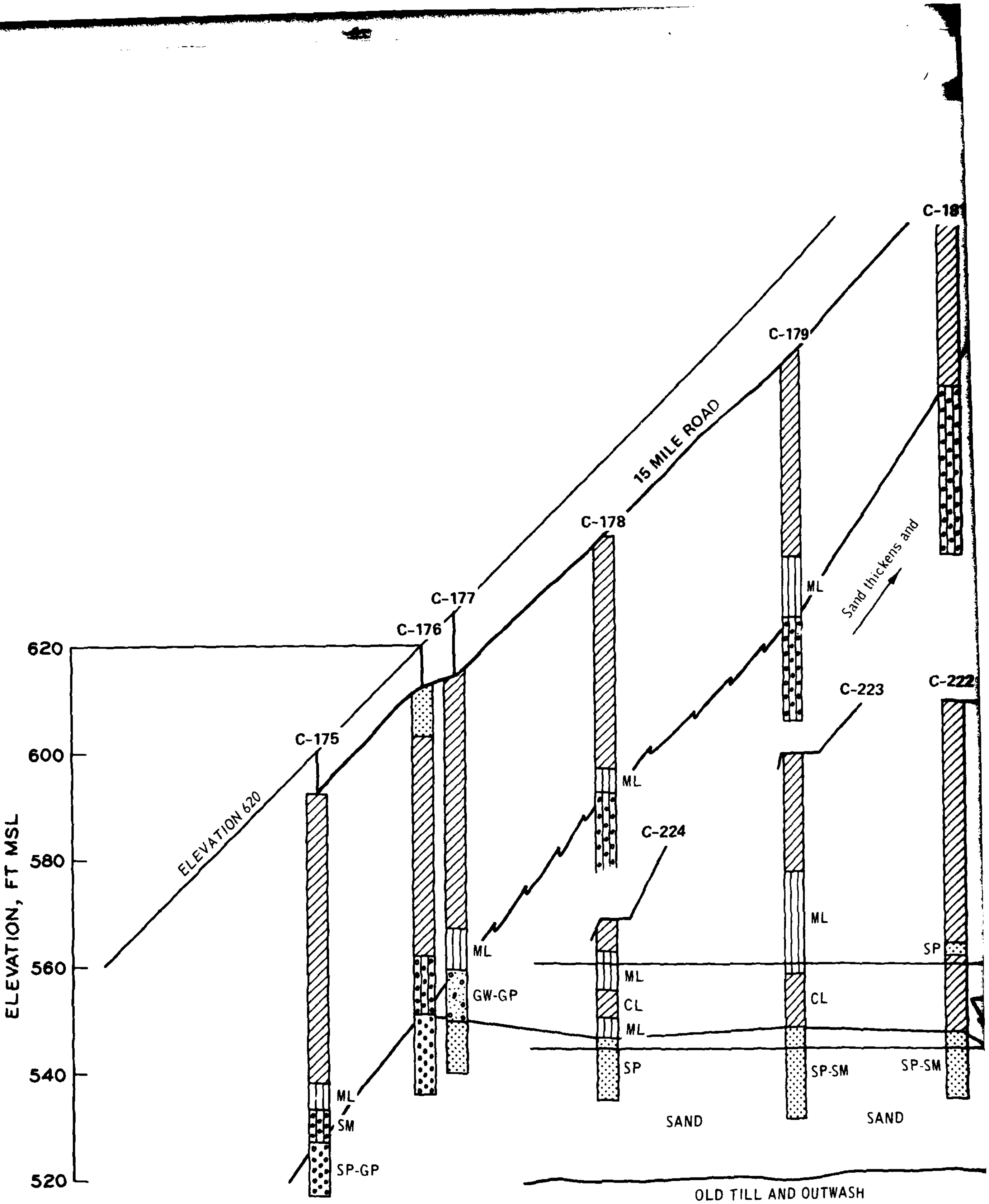
41. Land surface. The modern topography in the Detroit area is characterized as a relatively flat, featureless plain sloping gently southeastward toward Lakes St. Clair and Erie. The flat plain is the remnant of the glacial lake plain constructed at ancient higher lake levels. Northwest of the plain the topography is more rugged, dotted with irregularly shaped lakes and ridges and hills of till deposits of glacial moraines.

42. Local geology. The study site lies wholly within the glacial lake plain and is underlain by a complex series of lake clays and silts, gravelly clays (gravelly to rocky silty clays were encountered during tunnel excavation, para 52), sand and gravel deposits, and older till plains that predate the glacial lake deposits. The thickness of glacial deposits is approximately 150 ft (Twenter, 1975). A general geologic profile of the study area (based on contract PCI-3 boring logs) is shown in Figure 7. The clays are interpreted in this study as glacial lake deposits and reworked till (para 60). The glacial lakes formed in front of the ice lobes and changed in size, shape, and depth with each of several advances and retreats of the last glacier. Deposits within the lake area are therefore intermixed and complex. The bedrock formation beneath the glacial deposits at the site is the Antrim Shale.

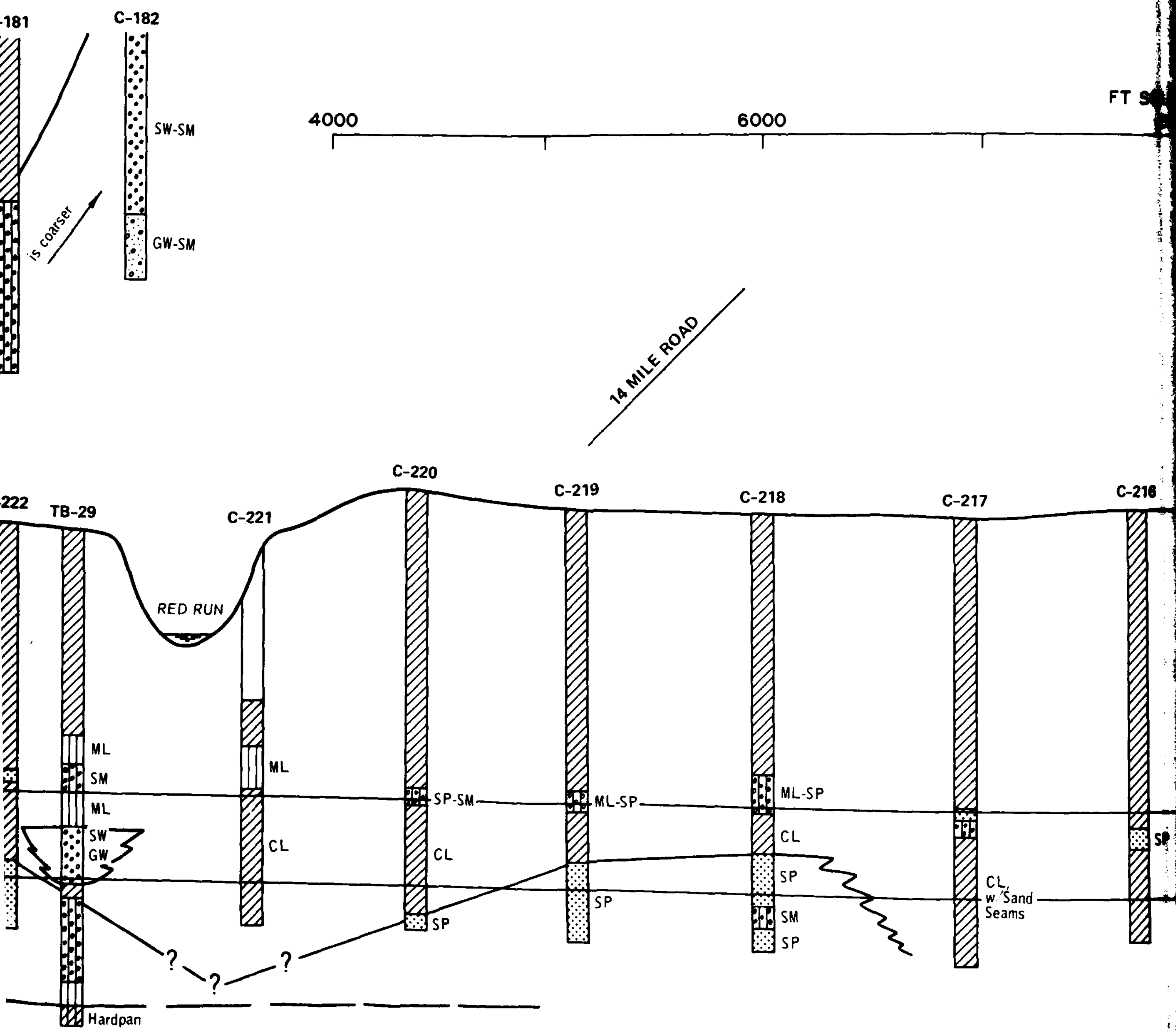
Groundwater

43. The source of the groundwater in southeastern Michigan is precipitation (Twenter,1975). The amount of precipitation that infiltrates the ground and recharges the groundwater reservoir is determined by the upper few feet of soil. If the soil contains a large amount of sand and silt, the infiltration rate will be high (8-12 in./hr). When soils are mostly clay, water does not penetrate rapidly and the infiltration rate will be less than 2 in./hr. Figure 8 shows a map of southeastern Michigan soils and their infiltration rates.

44. Occurrence of groundwater. The sediments in the areas shown as lakebeds (Figure 6) are a poor source of water (Twenter, 1975).



C-181
C-179
15 MILE ROAD
C-178
C-177
C-176
C-175
Sand thickens and
C-223
C-222
C-224
620
600
580
560
540
520
ELEVATION, FT MSL
ELEVATION 620
ML
GW-GP
SM
SP-GP
CL
SP
SP-SM
SAND
OLD TILL AND OUTWASH



NOTE: SEE APPENDIX A FOR LEGEND SHEET

GENERAL GEOLOGIC PROFILES BASED
ON CITY OF DETROIT CONTRACT
PCI-3 BORING LOGS

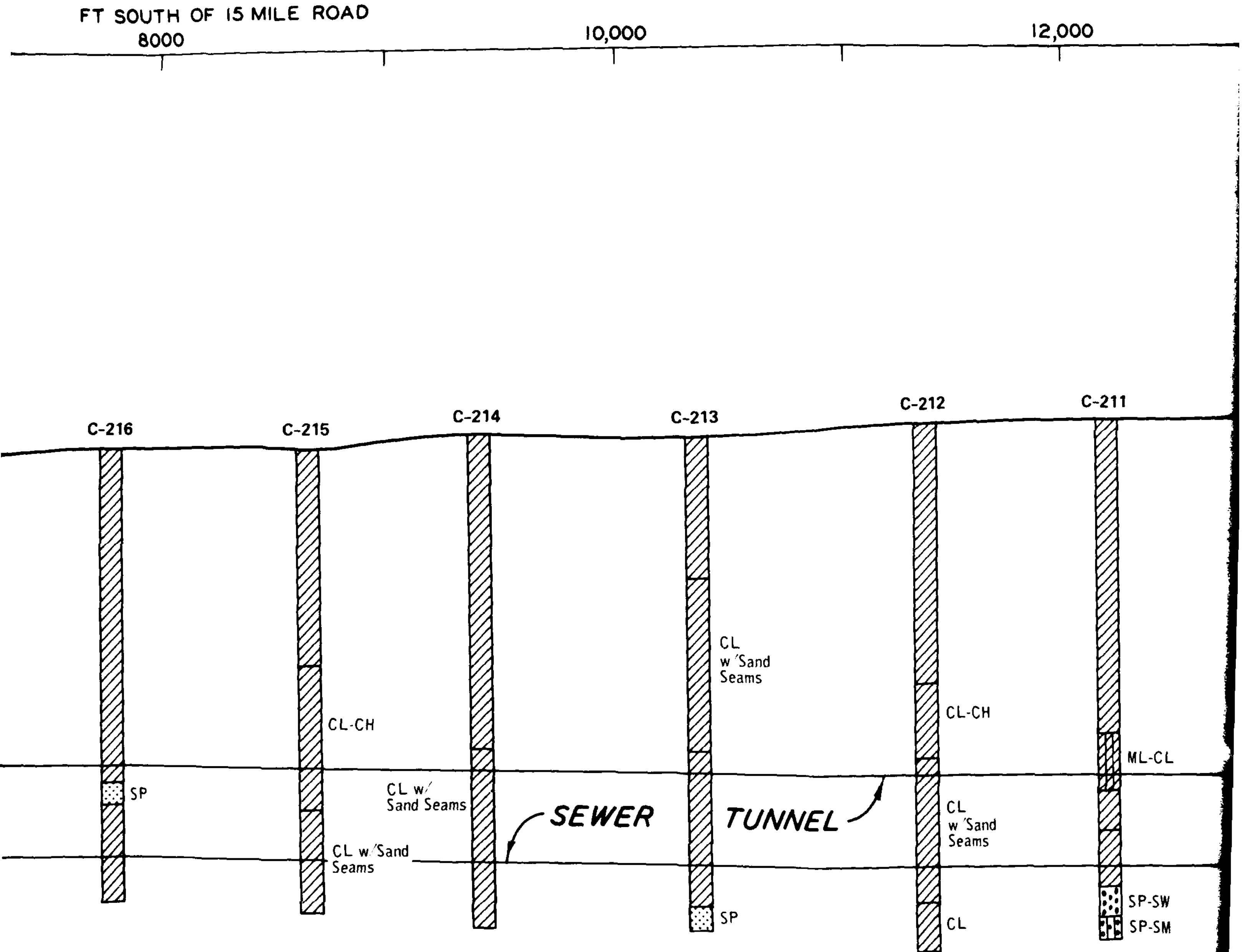


Figure 7. Site geologic p[...]
15 Mile Road/Edison Corrid[...]
Failure Study

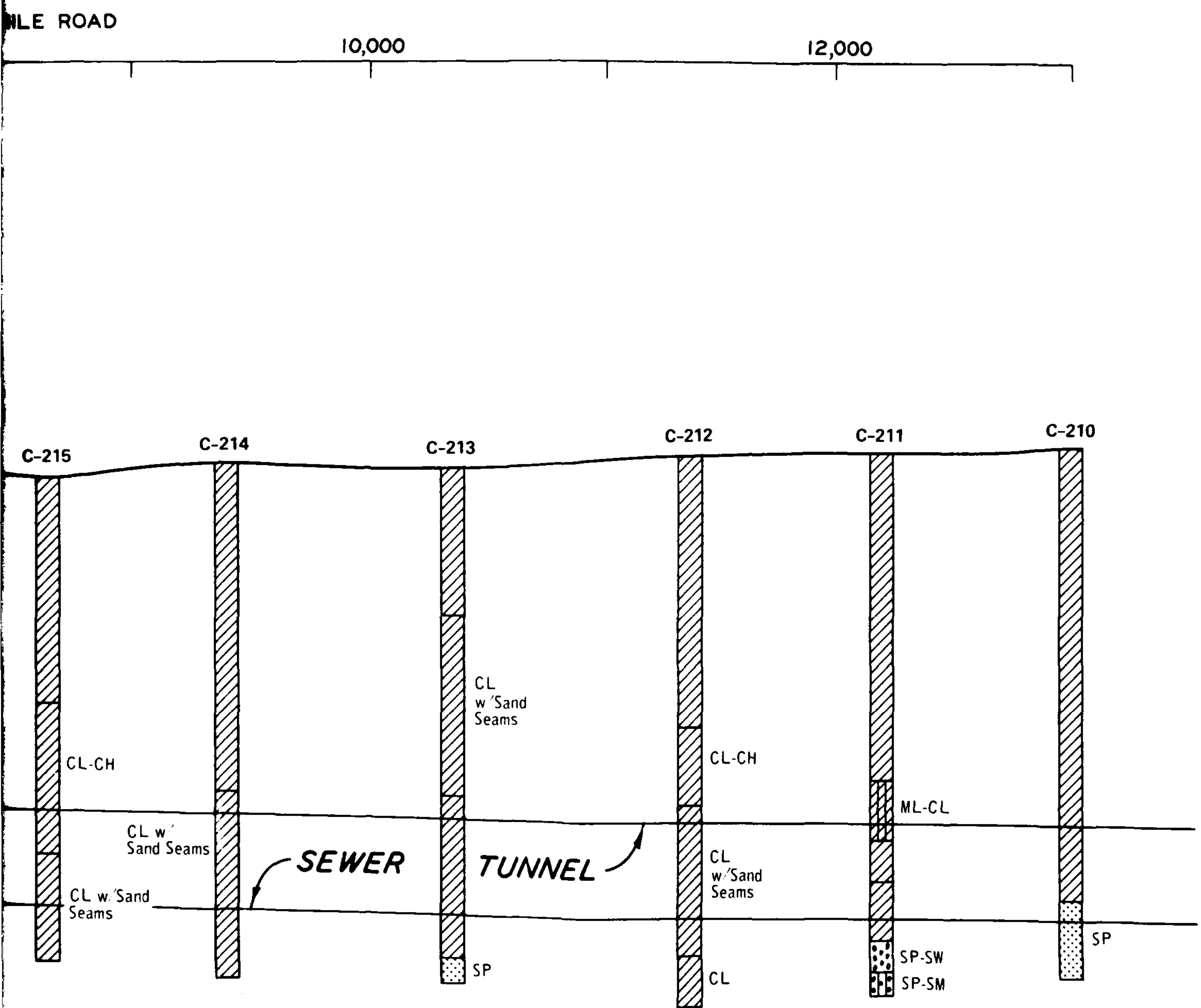


Figure 7. Site geologic profiles, 15 Mile Road/Edison Corridor Tunnel Failure Study

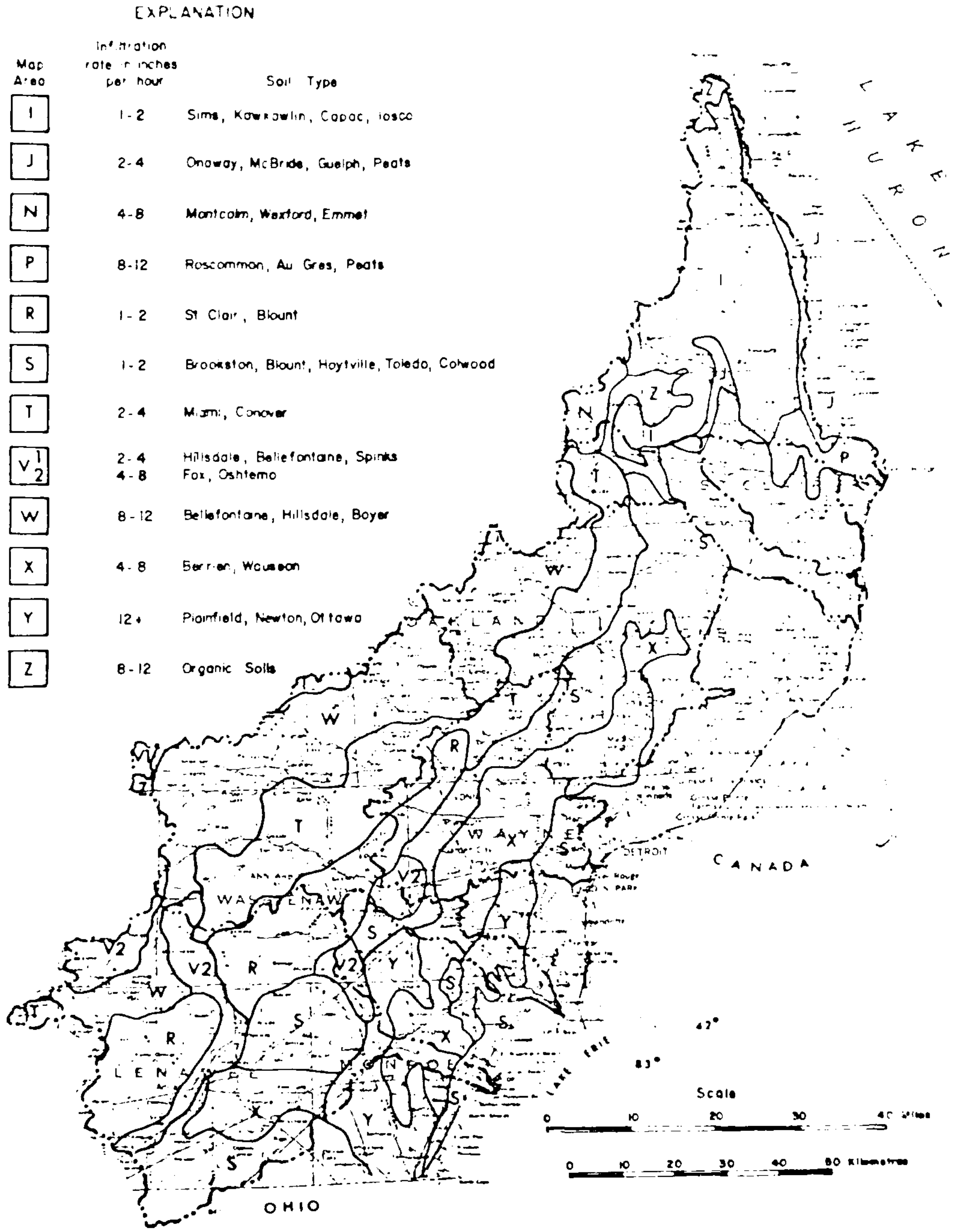


Figure 8. Soils and their infiltration rates, southeast Michigan (Twenter, 1975)

They are composed primarily of clay-rich materials and are not very permeable. A series of hills and ridges composed of sand and gravel 25 to 30 ft thick rest on top of the clay-rich materials in some areas. These hills and ridges were formed as beaches, terraces, and river deltas by wind, waves, and stream action during the late Wisconsin Glaciation (Wisler, 1952). They are very porous and absorb a large amount of the precipitation. The water seeps downward to the water table where it contacts the thick layers of lake clays and inter-bedded deposits of sand and gravel which are buried beaches, bars, and deltas associated with former lake levels (Mozola, 1969).

45. Recharge. The interbedded deposits of sand and gravel are probably gradational with the matrix and are probably interconnected to each other (Mozola, 1973). These deposits are partially confined and develop artesian systems. The system is recharged by either an outcrop of an aquifer at the surface (no matter how tortuous the route to the confined aquifer) or by crossbed leakage. "Permeability in glacial till is relative; a large cross section of clay till could pass as much water as a small cross section of sand. A large sand and gravel body, even if enclosed by till of low permeability can still function as a good aquifer because of the extensive contact with the surrounding till" (Mozola, 1969).

46. Local groundwater. At the site, the glacial sediments consist of about 60 ft of gravelly clays overlying some 30 ft of sorted sands and silts (Figure 7). The sand unit lies on top of glacial till and outwash of an earlier glacial age. This last unit is inferred to extend to a depth of 150 ft where it contacts the Antrim Shale. The sand unit is interpreted as a beach deposit due to its characteristic sorting and large areal extent. This unit is probably interconnected in the subsurface with the large glacial lake sand deposits which are located less than 2000 ft to the southwest (Figure 6). A minor amount of recharge water comes from crossbed leakage from the clay deposits due to the large areal extent of the beach deposit. Some recharge could also be contributed by seepage from the Red Run River at the site. In addition, as indicated by Figure 7, the sand thickens and is coarser

to the east of the site along 15 Mile Road. Preconstruction borings just east of the Red Run (east of C-182 on Figure 7) indicate that the sands extend up to the ground surface. Farther east to Lake St. Clair, deep glacial clays separate infrequent narrow beach sand deposits and it is considered unlikely that the St. Clair lake is connected to the sands at the tunnel site.

Preconstruction conditions

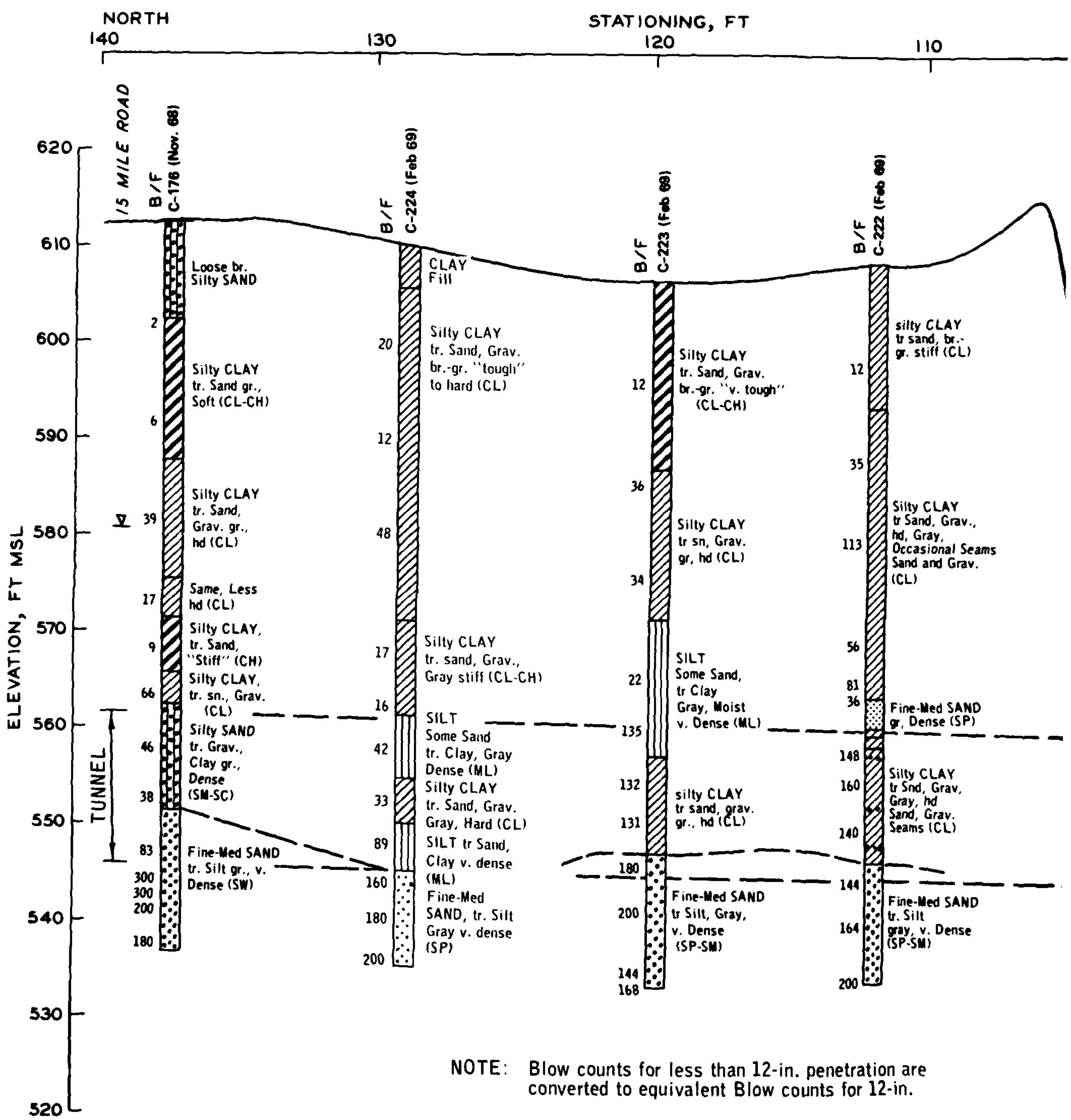
47. Preconstruction boring logs C-176, C-224, C-223, and C-222 cover the tunnel study area from Stas 137 to 112. A profile of these boring logs, shown in Figure 9, indicates variable strata of gray hard silty clays and silts along the tunnel and the top of a strata of very dense, fine to medium sand, along the base of the tunnel. Blow counts increase significantly in sands below elevation 550 to 545 and generally range from 150 to 180 in the sand. Minimum blow counts below elevation 560 are significantly higher in the silty clays and silts in borings C-223 and C-222 than in C-176 and C-224. Although all boring logs indicated only a trace of gravel in the clays and silts above the tunnel, the high blow counts in borings C-224 and C-222 could be the result of higher gravel content.

48. Soil properties. Soil properties were determined by DWSD for small samples (1.4-in. diam) taken with a drive sampler in borings C-176 and C-222. A summary of water contents, wet densities, unconfined shear strength, and transverse shear strength shown in Figure 10 (boring log legend is in Appendix A) indicate the following:

a. Samples from both borings had similar water contents, ranging from about 30 percent in upper clays to about 12 percent below elevation 585. Wet densities ranged from about 130 to 150 pcf below elevation 590. Densities of samples from C-222 were generally higher than for samples from C-176.

b. Unconfined shear strengths ranged from about 200 psf (0.1 tsf) to about 10,000 psf (5 tsf) with generally higher values for samples from C-222. Samples below elevation 563, logged as silty sand and fine to medium sand in boring C-176 (Figure 10), were classified as silt and fine sand, very clayey silt and fine sand, and silt and fine sand when tested in the DWSD laboratory. Transverse shear resistance (Housel, 1970)

Blank



NORTH
STATIONING, FT
140
130
120
110
ELEVATION, FT MSL
620
610
600
590
580
570
560
550
540
530
520
15 MILE ROAD
B/F C-176 (Nov. 68)
B/F C-224 (Feb 69)
B/F C-223 (Feb 69)
B/F C-222 (Feb 69)
TUNNEL
Loose br. Silty SAND
Silty CLAY tr. Sand gr., Soft (CL-CH)
Silty CLAY tr. Sand, Grav. gr., hd (CL)
Same, Less hd (CL)
Silty CLAY, tr. Sand, "Stiff" (CH)
Silty CLAY, tr. sn., Grav. (CL)
Silty SAND tr. Grav., Clay gr., Dense (SM-SC)
Fine-Med SAND tr. Silt gr., v. Dense (SW)
2
6
39
17
9
66
46
38
83
300
300
200
180
CLAY Fill
Silty CLAY tr. Sand, Grav. br.-gr. "tough" to hard (CL)
Silty CLAY tr. sand, Grav., Gray stiff (CL-CH)
SILT Some Sand tr. Clay, Gray Dense (ML)
Silty CLAY tr. Sand, Grav. Gray, Hard (CL)
SILT tr Sand, Clay v. dense (ML)
Fine-Med SAND, tr. Silt Gray v. dense (SP)
20
12
48
17
16
42
33
89
160
180
200
Silty CLAY tr. Sand, Grav. br.-gr. "v. tough" (CL-CH)
Silty CLAY tr sn, Grav. gr, hd (CL)
SILT Some Sand, tr Clay Gray, Moist v. Dense (ML)
silty CLAY tr sand, grav. gr., hd (CL)
Fine-Med SAND tr Silt, Gray, v. Dense (SP-SM)
12
36
34
22
135
132
131
180
200
144
168
silty CLAY tr sand, br.-gr. stiff (CL)
Silty CLAY tr Sand, Grav., hd, Gray, Occasional Seams Sand and Grav. (CL)
Fine-Med SAND gr, Dense (SP)
Silty CLAY tr Snd, Grav, Gray, hd Sand, Grav. Seams (CL)
Fine-Med SAND tr. Silt gray, v. Dense (SP-SM)
12
35
113
56
81
36
148
160
140
144
164
200
NOTE: Blow counts for less than 12-in. penetration are converted to equivalent Blow counts for 12-in.

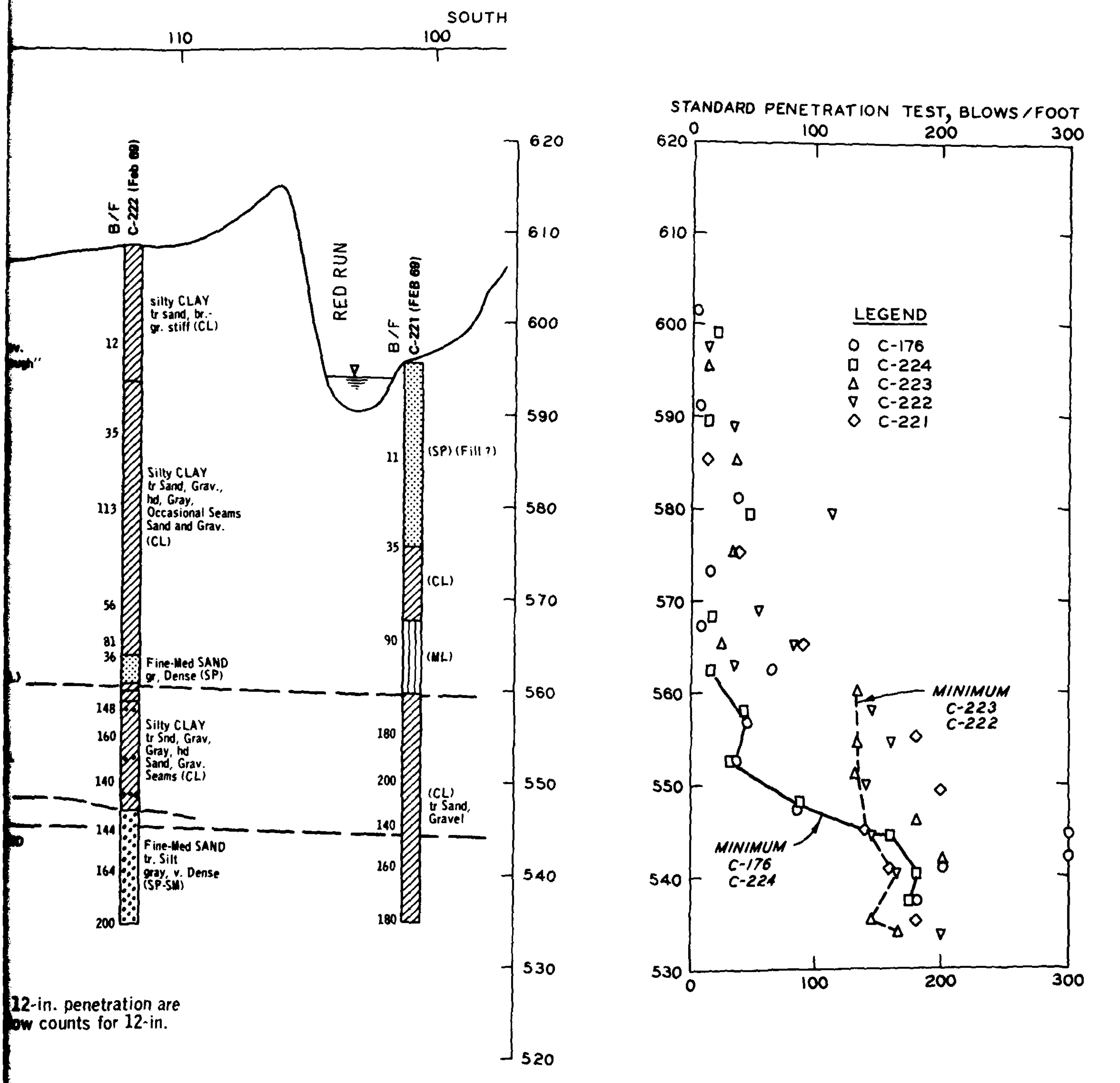
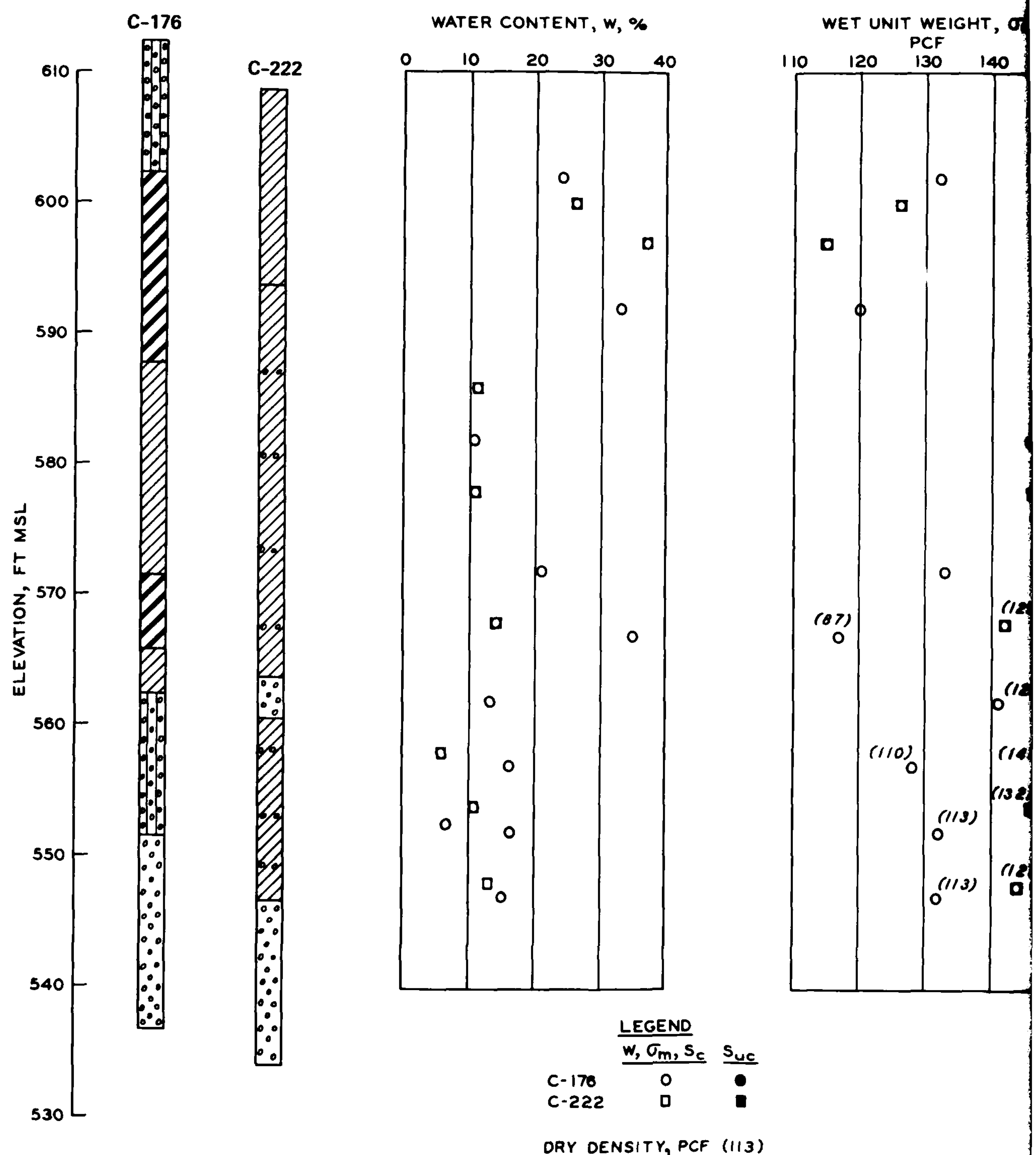


Figure 9. Preconstruction geologic profile, 15 Mile Road/Edison Corridor Tunnel Failure Study



C-176
C-222
ELEVATION, FT MSL
610
600
590
580
570
560
550
540
530
WATER CONTENT, W, %
0
10
20
30
40
WET UNIT WEIGHT, σ
PCF
110
120
130
140
(87)
(110)
(113)
(113)
LEGEND
W, σm, Sc
Suc
C-176
C-222
DRY DENSITY, PCF (113)

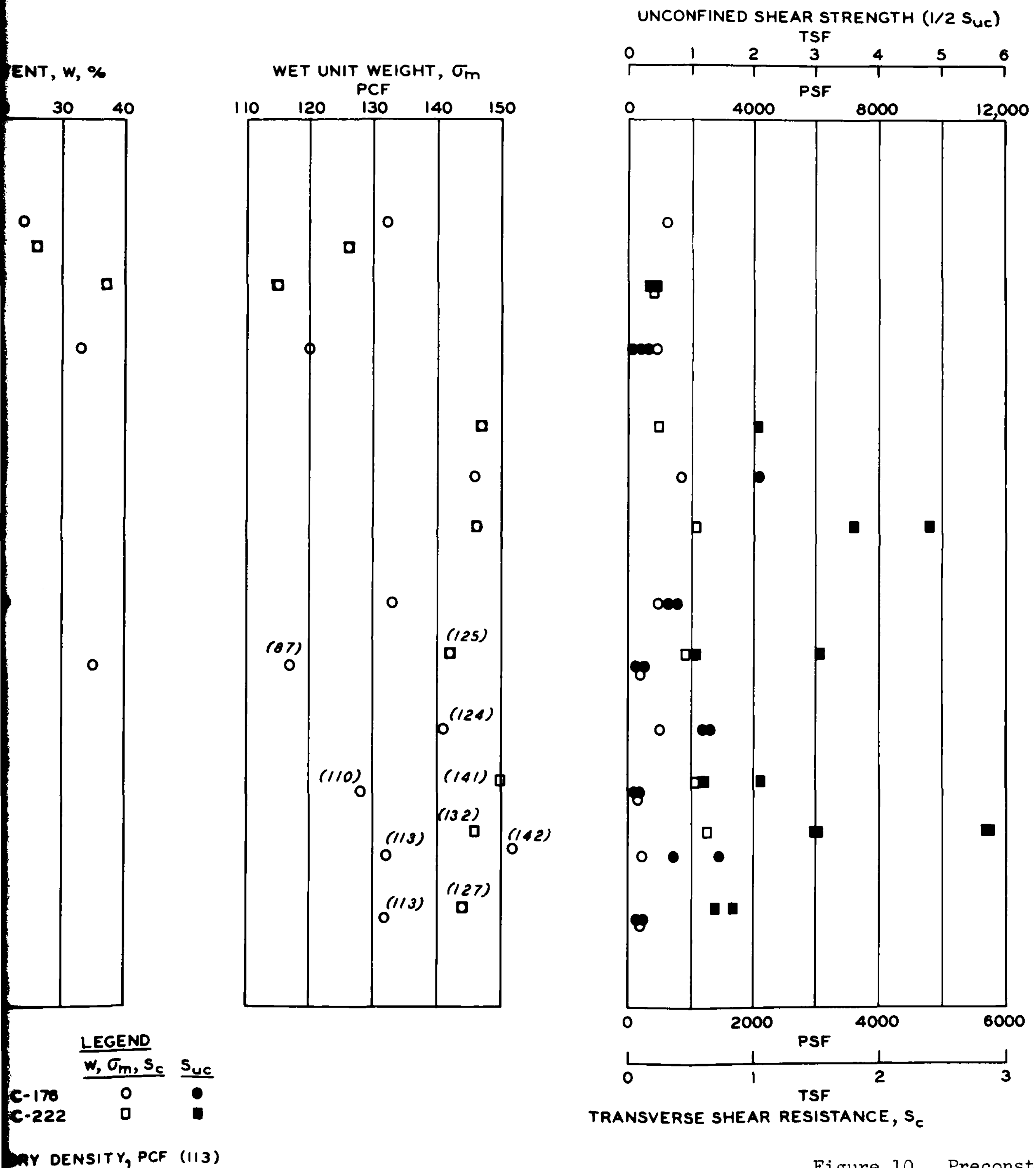


Figure 10. Preconstruction soil properties, 15 Mile Road/Edison Corridor Tunnel Failure Study

values were generally less than 1,400 psf (0.7 tsf) and about equal to one-fourth of the unconfined shear strength. The values for samples from C-176 were about one-half of those for samples from C-222.

c. The blow counts and soil property values indicate that strata at and just above the tunnel depths north of about Sta 128 are relatively softer than south of this station. No gradation or soil characteristics other than blow counts for the fine to medium sands along the tunnel base were measured. Also, the extent of these sands to the south of about Sta 111+50 to the Red Run were not defined. Consequently, their influence on tunnel stability does not appear to have been considered.

49. Groundwater conditions. Boring C-176, in connection with contract PCI-8 was made using a hollow stem auger and no drilling water. Consequently, the groundwater elevation of about 581 observed in November 1968 is considered a reliable indication of artesian pressures in the fine to medium sand below elevation 552 (Figure 9). The preconstruction borings C-222 to C-224 along the PCI-7 study area were made using casing to shallow depths and drilling water. The shallow water levels observed in these borings before casing removal probably reflects the influence of drilling water and are not true groundwater levels. Consequently, artesian water pressures in the sands near the tunnel base were not defined south of Sta 137 to the Red Run.

Construction conditions

50. Tunnel construction conditions important to geotechnical considerations were dewatering; soils and seepage conditions encountered during mining; times between initial lining and concreting; location of construction joints, cold joints, and water stops; and leakage after concreting. These aspects are discussed below.

51. Dewatering. Deep construction dewatering wells were installed between Stas 136+33 and 105+80 from 20 October to 14 December 1970 as summarized in Table 5 from DMWD daily reports. Daily inspector reports also indicated initial installation of these wells as shown in Table 6 (first two items). The wells were located 25 ft east of the

tunnel centerline at a spacing of 100 ft from Sta 134 to Sta 114, and extended some 30 ft into the sand underlying the tunnel invert. From Stas 112+20 to 105+80, the spacing ranged from 10 ft to 100 ft (average spacing of 46 ft). The depth of water shown in Table 5 was the depth that free water was first encountered during drilling and does not reflect the artesian pressures that would be indicated if water level readings in the borings had been taken several days after drilling. The wells apparently were operated until completion of construction in August 1972 as indicated from inspector's reports summarized in Table 6 (last three items). No records were found of the dewatering level maintained during construction. Consequently, it is assumed that the ground around the tunnel was dewatered to the depth of the mined invert (since no seepage was noted during mining until Sta 125+81, Table 7), and that ground pressures during construction corresponded to wet unit weights for the soils (no buoyant effect) which ranged from 130 to 150 pcf. After construction and removal of dewatering, the artesian groundwater pressure in the sand along the base of the tunnel was free to increase to preconstruction levels of 5 ft to 15 ft above the tunnel crown, depending on recharge from rains and surface infiltration (discussed in para 45 on groundwater) and watertightness of the completed tunnel.

52. Soils encountered during mining. Soils encountered during mining from Sta 135+83 to 103+93, as described in daily inspector's shift reports, are listed in Table 7. Although the predominant soil type was gray, sandy clay and clay, hard to very hard with varying amounts of rock, frequent changes occurred in the amount of sand, gravel, and rocks. Gravel and sand with clay was noted for Stas 134+35 to 133+83. Hard gray sandy clay from Stas 132+95 to 132+85 changed to gray sand with clay from 132+50 to 132+25 and gray sand with very hard gray sandy seams from 132+70 to 127+76 changed to clay, very rocky from 127+76 to 127+21. Gravel, sand, and sand strata were indicated at a number of locations as indicated by underlined words in Table 7. Except for the rocky condition, the soils excavated at the location of preconstruction borings generally agreed with the boring logs. Running sand

strata as mining neared the Red Run were encountered south of Sta 109+16 (see Excavation Summary, Table 1).

53. Seepage during mining. No indication of seepage into the tunnel was noted in inspector's shift reports until Sta 125+81 where some water started to appear. Other indications of seepage are highlighted by dashed underlining in Table 7. Seepage increased south of Sta 109 as mining approached the Red Run. Water seepage and caving ground south of Sta 109 are noted in the Excavation Summary, Table 1. The caving and increased seepage near the Red Run correlate with the sand layer and sand strata shown in the log for boring C-222 (Figure 9).

54. Tunnel lining. The conditions during tunnel lining construction important to geotechnical considerations are discussed below.

a. Ribs and lagging. Steel ribs (5WF16) on 5-ft centers and 4-in thick wood lagging (between flanges of the ribs) were erected within a shield behind the mining machine and jacked against the tunnel soil surface as the machine pushed ahead. Because of the numerous rocks (as large as 4 ft) encountered during mining (many required blasting for removal through the mining wheel, as noted in the Excavation Summary, Table 1), the tunnel surface was probably irregular with depressions and hard rock spots. Thus, it is possible that numerous small voids existed between the wood lagging and the tunnel soil wall. Voids, if present, would increase the available seepage paths for water to flow from the soil through spaces in the lagging and through any open joints in the concrete lining after construction and removal of the dewatering system. The construction sequence from Tables 1 and 2 indicates that the concrete lining was placed some four months after the ribs and lagging were installed. This time allowed redistribution of ground pressures around the tunnel, probably to an all around pressure less than the overburden pressure.

b. Concrete lining. The location of construction joints, waterstops, and dates of pours are summarized for Stas 136 to 107 in Figure 11. The 16-in. thick (nominal) concrete lining was poured in 105 ft lengths. South of Sta 119, construction joints with waterstops existed at 105 ft intervals. No waterstops were installed north of Sta 119 (Table 1). Distance from concrete form to rib and lagging (liner) condition

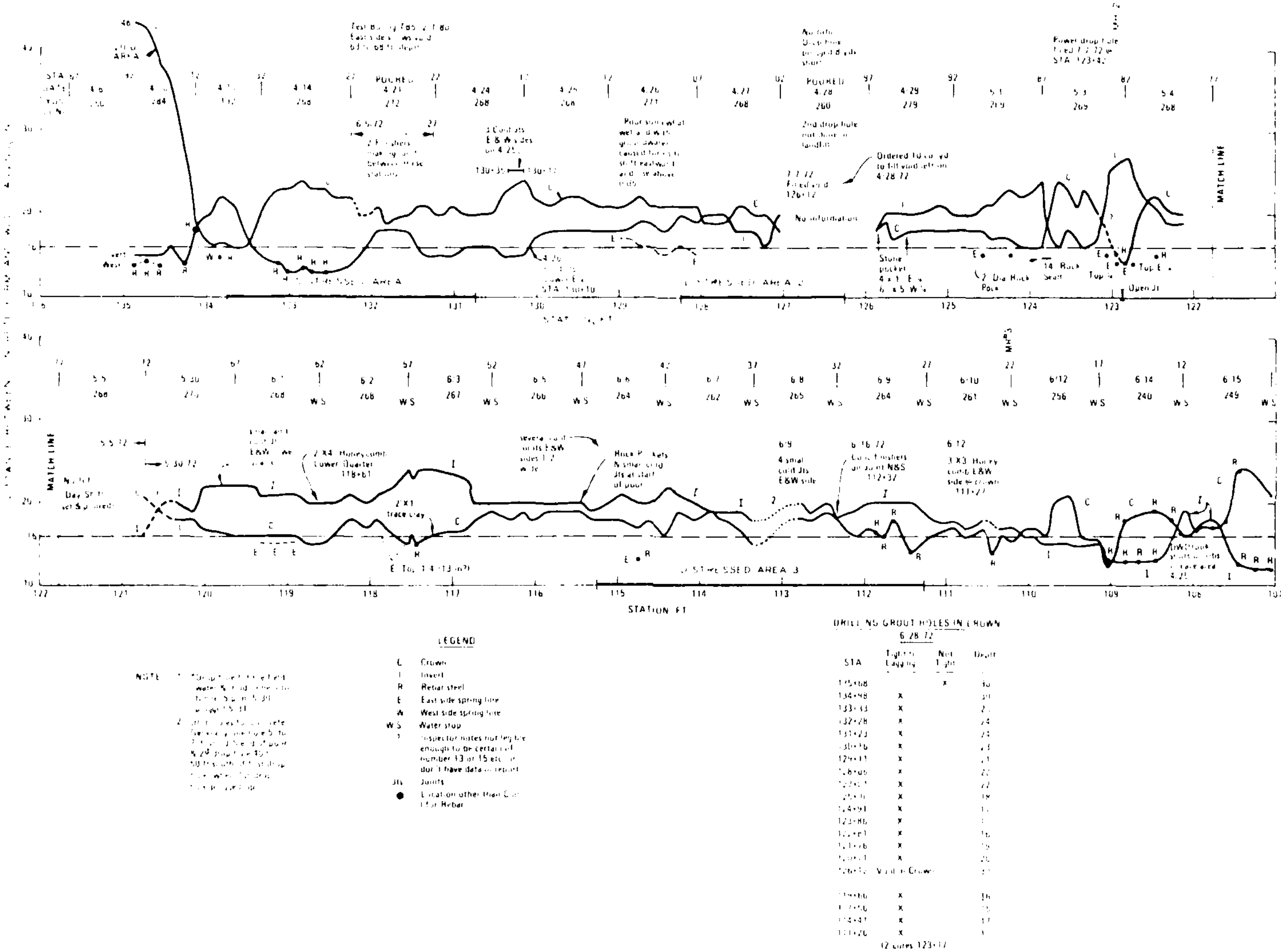


Figure 11. Construction conditions of lining

after pour, and other pertinent notes from City inspectors daily shift reports are also shown in Figure 11. Locations of distressed areas are shown for subsequent reference. The construction joints, cold joints, concrete finishing (in both distressed and nondistressed areas) that could effect the water-tightness and potential for soil infiltration after construction and removal of dewatering could influence the stability of the tunnel. The measured distances to the lining from the concrete forms are an indication of the variation in tunnel dimensions after four months and just prior to concrete placement. This variation could have an influence on ground pressure distribution along the tunnel. The tabulation of grout holes drilled in the crown to check tightness of concrete against lagging is included in Figure 11. The results indicated only two locations where either the concrete was not tight against the lagging or a void existed (Stas 136+68 and 126+12). The void at Sta 126+12 was filled on 7 July 1972 as was a power drop hole at Sta 123+42.

c. Entry of water. During concrete placement, the drop hole at Sta 118+57 pierced a tile field. Water and mud entered the tunnel at this location (shift inspector's daily reports for 30 May to 4 June 1972). Any water that leaked around the outside of the lagging could have eroded soil from around the upper section of the tunnel and through the lagging near the springline, causing a potential weak spot in the tunnel.

d. Tunnel leakage. Notes in City inspector's daily reports indicated concrete finishers were "finishing joints and leaks down to Sta 103+95" on 28 June 1972, "continued to finish tunnel north of cofferdam shaft and stop leaks" on 29 June 1972, "finishers continued to stop leaks and finish joints at various places north of cofferdam shaft" on 5 July 1972. How far north this work extended was not recorded. It appears that leakage at joints occurred after concrete liner placement, but specific locations and type of leakage (i.e., clear water or water and fine sand or silt) is unknown for the study section of the tunnel.

e. Effect of Red Run cofferdam. During construction, a cave-in at the Red Run (Table 1, Excavation Summary) required a cofferdam which had significant leakage and resulted in a shaft which was later backfilled. No information was found on compaction of backfill. It is not known how impervious the backfill was. It is possible that increased leakage from the Red Run through the backfill could have contributed to the

groundwater recharge and increased the groundwater levels at the site.

Postconstruction conditions

55. Once the dewatering system was removed after construction, the groundwater could return to normal levels. Preconstruction boring C-176 for PCI-8 indicated a groundwater level at elevation 581, some 35 ft above the tunnel invert. The 35-ft head would cause a water pressure of 15 psi at the invert and 8 psi at the crown. Any open joints in the tunnel would be subject to leakage and possible infiltration of fine sand and silty sand where such strata existed at a leaking joint. No observation or producing wells exist in the local area and thus a direct measurement of groundwater levels since construction does not exist. Since surface infiltration in the region is the primary source of recharge for the beach sand along the tunnel base, variations in rainfall and Red Run stages could effect the maximum groundwater level especially during unusually wet seasons. Local rainfall and river discharge records were reviewed from 1972 to the present and the results are summarized below. Possible seismic activity in the area was also considered. The only activity during 1972-1980 was a small tremor of magnitude 3.4 centered at Point Pelee, Ontario, some 48 miles from the tunnel site.

56. Rainfall records. Rainfall records through 1978 for the Detroit City airport, about 9 miles south of the site, indicated higher rainfall in 1973, 1975, and 1976 than in 1974 and 1977, based on yearly totals. The wettest year was 1976.

57. Red Run and Clinton River stages and discharges. Gage stations on the Red Run were not established until October 1979. However, discharge records for a gage on the Clinton River near the Garfield Road bridge provided data from 1972 through October 1979. This gage is one mile from the exit of the Red Run into the Clinton and some two miles from the tunnel site. The number of consecutive days for discharges over 1000 cfs was used as a measure of significant runoff from long-term rainfall periods. The comparison showed 16, 18, 8, and 15 continuous days of 1000 cfs or greater discharge for 1973-1976

and only 3 to 4 continuous days for 1977-1979. Thus much higher runoff, suggesting much more rainfall, was apparent for 1973-1976 than for 1977-1979. The 1973-1976 long-term rainfall periods would allow greater infiltration and recharge of the groundwater and produce relatively higher groundwater levels than in 1977-1979. The nearest observation wells to the site are in Oakland County. Water levels for a well west of Pontiac (Huffman, 1979) in glacial deposits showed maximum extreme levels in 1974 and 1976. The variation between the monthly high and low for 1976 was 2 ft and slightly less than 2 ft for monthly lows for 1973-1977. Thus, even in wet years, the groundwater rise may be relatively small.

58. Red Run borings. Borings for the Corps of Engineers made in September 1978 indicated a groundwater level at a depth of 40 ft in a cased boring (RR-43-78) some 270 ft east of tunnel Sta 111+10. This groundwater depth corresponds to an elevation of 565 near the crown of the tunnel. Several other borings along this reach also indicated a groundwater level at the 40 ft depth. This level was during the drier season and the level during the spring may have been higher.

Postfailure conditions

59. The subsurface soil stratification and soil conditions after tunnel distress indicated by test borings made for the DWSD in Feb-Mar 1980 were confirmed by the WES borings. The locations of all test borings and WES borings are shown by four plan plots along the centerline in Figure 12, in relation to the distressed areas. The post failure geotechnical conditions based on these borings, tunnel drill holes, testing and sampling, laboratory soil testing, and visual observations during site visits are described in this section.

60. Soil stratification and in situ conditions. Figure 13 is a detailed stratigraphic profile along and near the tunnel centerline from approximately 15 Mile Road south to near the Red Run. The "as-built" tunnel profile is shown to indicate the elevations of different soil strata relative to the tunnel before distress. The profile was constructed with data selected from three sets of borings:

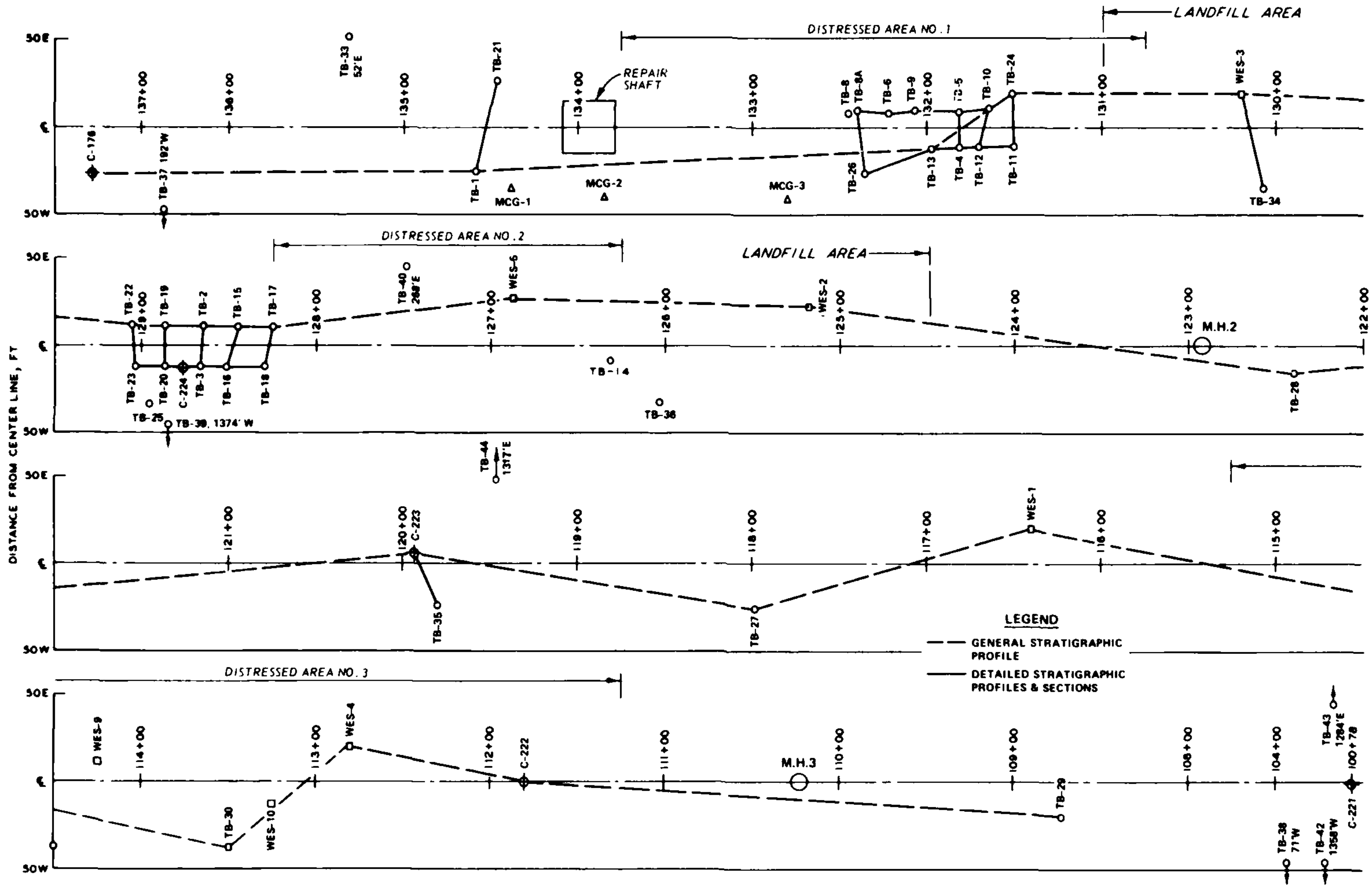
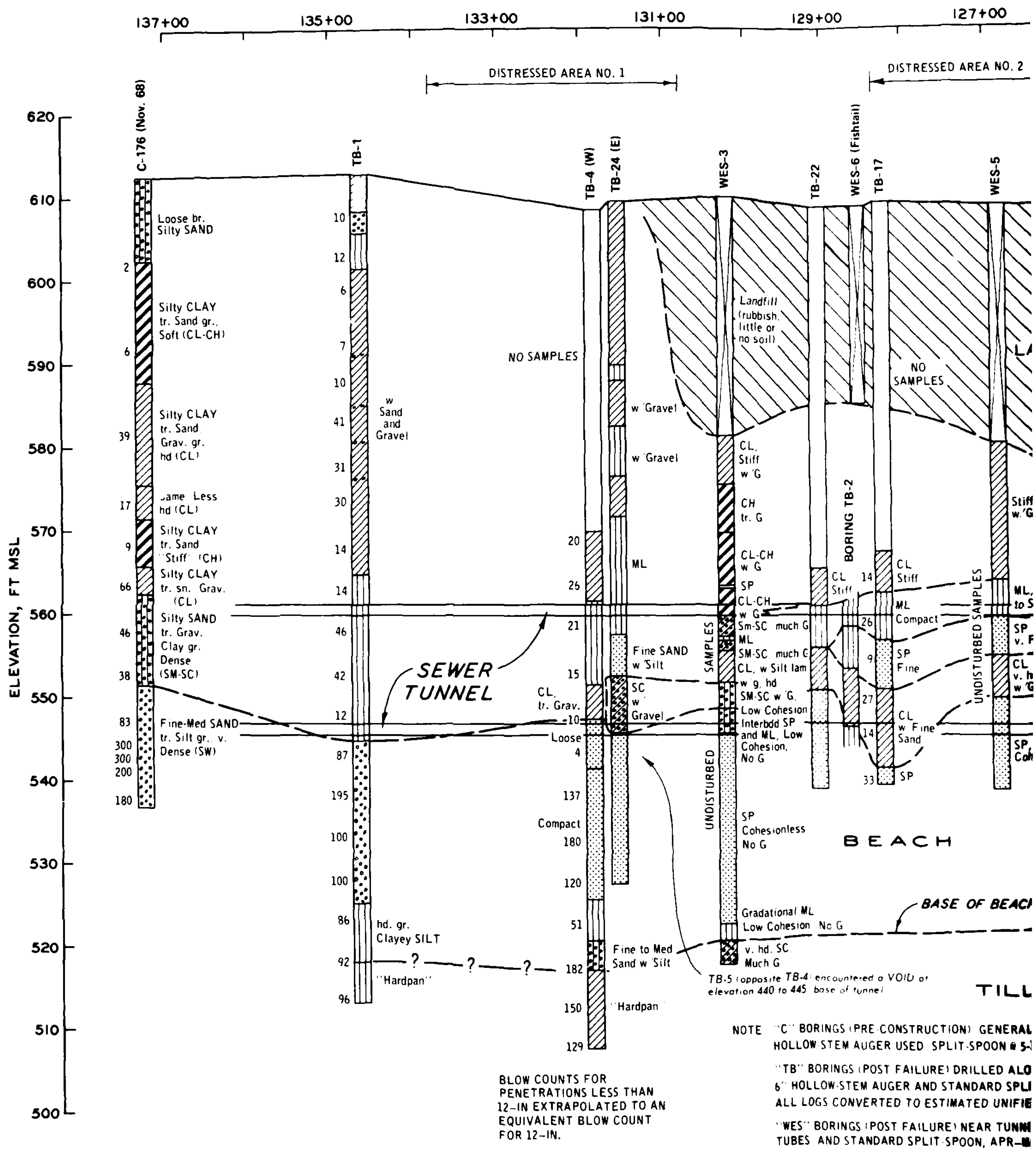
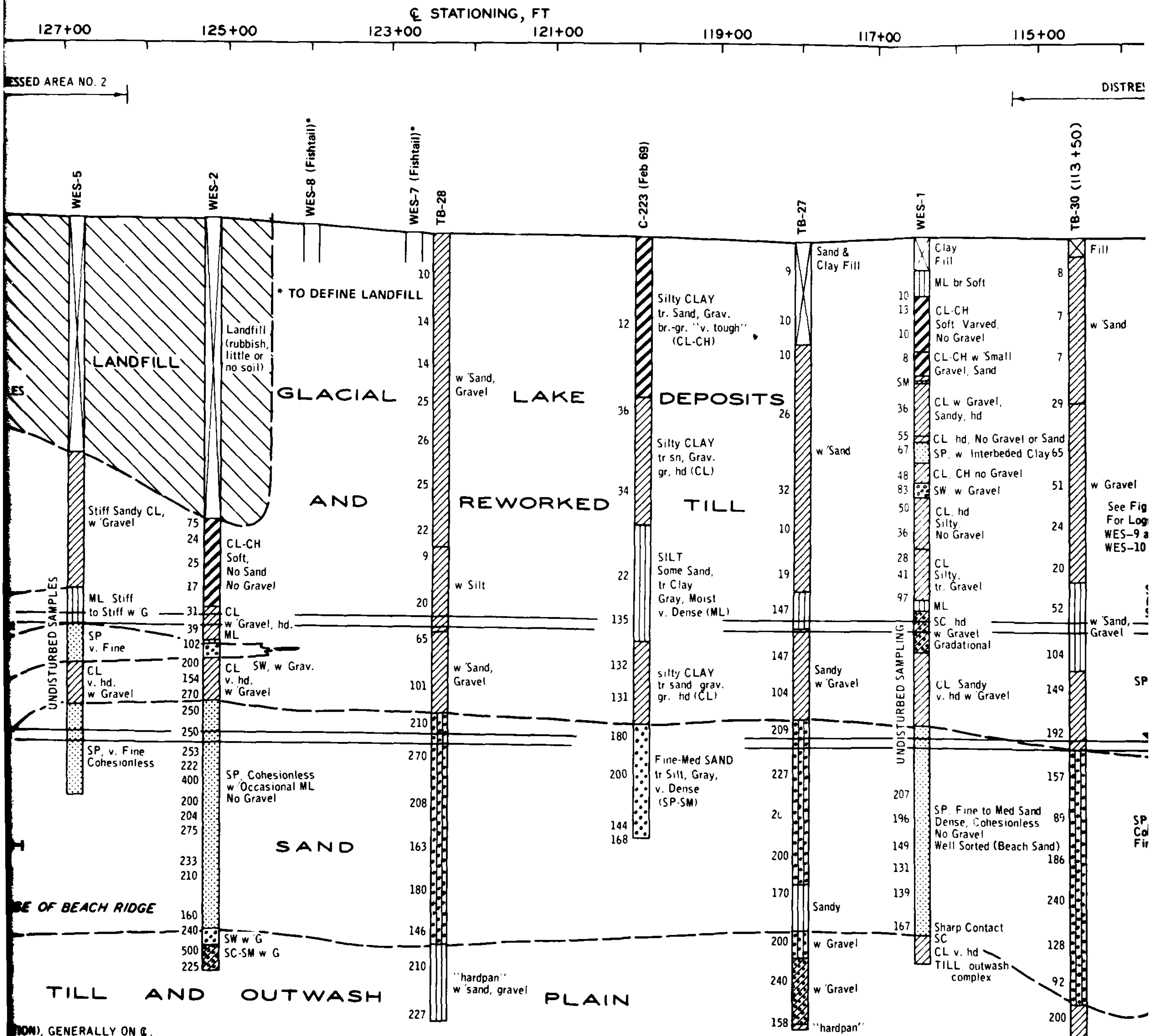


Figure 12. Location of all borings used in developing subsurface stratification and soil conditions, 15 Mile Road/Edison Corridor Tunnel Failure Study



137+00
135+00
133+00
131+00
129+00
127+00
DISTRESSED AREA NO. 1
DISTRESSED AREA NO. 2
ELEVATION, FT MSL
620
610
600
590
580
570
560
550
540
530
520
510
500
C-176 (Nov. 68)
TB-1
TB-4 (W)
TB-24 (E)
WES-3
TB-22
WES-6 (Fishtail)
TB-17
WES-5
BORING TB-2
Loose br. Silty SAND
Silty CLAY tr. Sand gr., Soft (CL-CH)
Silty CLAY tr. Sand Grav. gr. hd (CL)
Same Less hd (CL)
Silty CLAY tr. Sand "Stiff" (CH)
Silty CLAY tr. sn. Grav. (CL)
Silty SAND tr. Grav. Clay gr. Dense (SM-SC)
Fine-Med SAND tr. Silt gr. v. Dense (SW)
w Sand and Gravel
hd. gr. Clayey SILT
"Hardpan"
NO SAMPLES
w Gravel
ML
Fine SAND w Silt
SC w Gravel
CL, tr. Grav.
Loose
Compact
Fine to Med Sand w Silt
"Hardpan"
Landfill (rubbish, little or no soil)
CL, Stiff w G
CH tr. G
CL-CH w G
SP
CL-CH
w G
Sm-SC much G
ML
SM-SC much G
CL, w Silt lam
w g, hd
SM-SC w G
Low Cohesion
Interbdd SP and ML, Low Cohesion, No G
SP Cohesionless No G
Gradational ML Low Cohesion No G
v. hd. SC Much G
UNDISTURBED SAMPLES
CL Stiff
ML Compact
SP Fine
CL w Fine Sand
SP
UNDISTURBED SAMPLES
SEWER TUNNEL
BEACH
BASE OF BEAC
TILL
TB-5 (opposite TB-4) encountered a VOID at elevation 440 to 445 base of tunnel
BLOW COUNTS FOR PENETRATIONS LESS THAN 12-IN EXTRAPOLATED TO AN EQUIVALENT BLOW COUNT FOR 12-IN.
NOTE "C" BORINGS (PRE CONSTRUCTION) GENERAL
HOLLOW STEM AUGER USED SPLIT SPOON @ 5-
"TB" BORINGS (POST FAILURE) DRILLED ALO
6" HOLLOW-STEM AUGER AND STANDARD SPLI
ALL LOGS CONVERTED TO ESTIMATED UNIFIE
"WES" BORINGS (POST FAILURE) NEAR TUNN
TUBES AND STANDARD SPLIT-SPOON, APR-



ION), GENERALLY ON ℄.
LIT-SPOON @ 5-10 FT INTERVALS (NX).
DRILLED ALONG SIDE TUNNEL.
TANDARD SPLIT SPOON, FEB–MAR 1980
ATED UNIFIED SOIL CLASSIFICATION SYSTEM.
NEAR TUNNEL, 6" ROCK BIT, 5" & 3" SAMPLE
POON, APR–MAY 1980

LEGEND

PROBABLE STRATIGRAPHIC CONTACT.

FACIES (GRADUAL) CHANGE OR INTERFINGERING CONTACT.

NUMBERS ON SIDE OF LOGS ARE BLOWS/FT

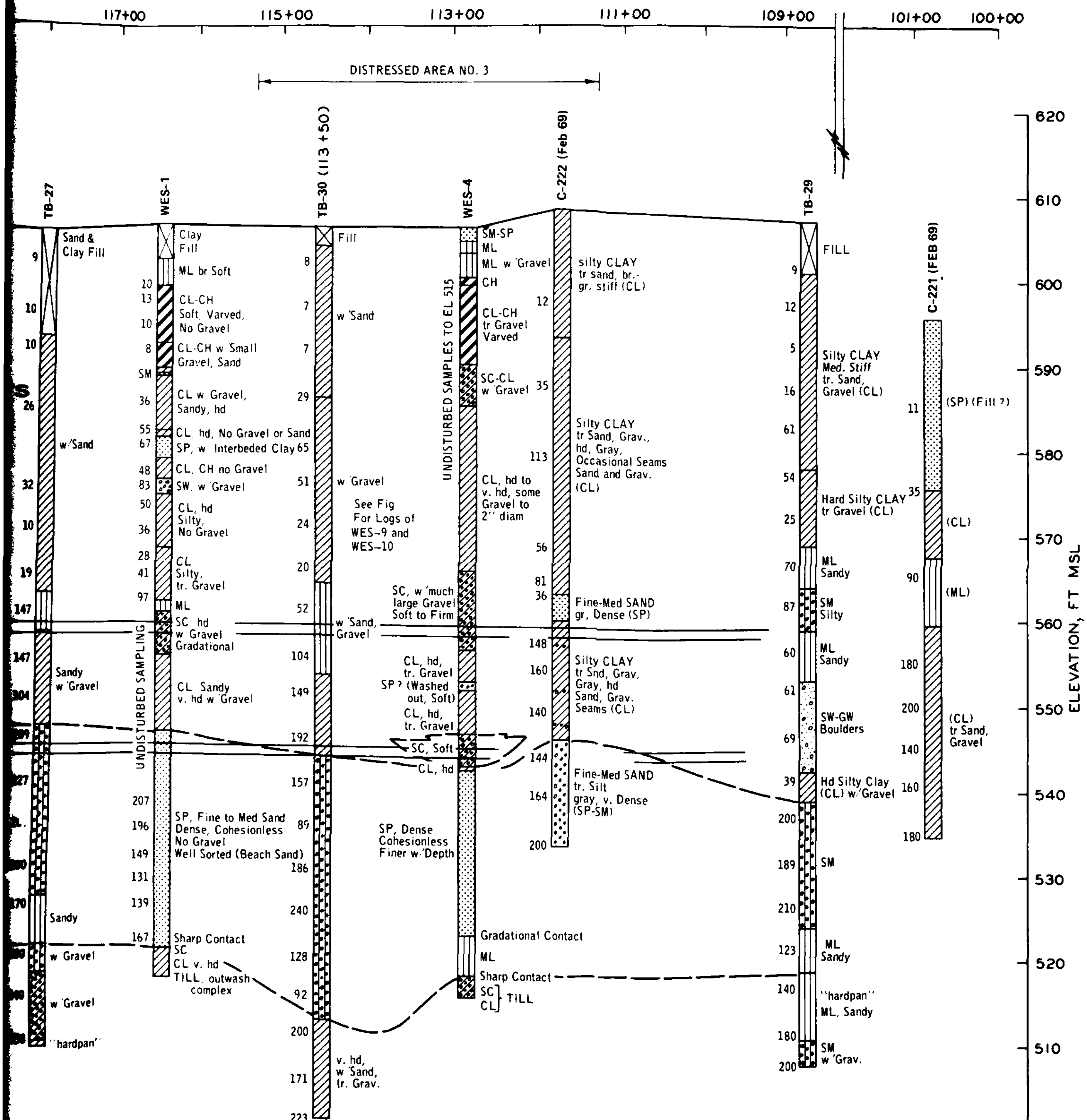


Figure 13. Detailed stratigraphic profile, 15 Mile Road/Edison Corridor Tunnel Failure Study

(1) "C" borings, emplaced along the centerline in 1969, prior to tunnel excavation and construction; (2) "TB" borings, emplaced alongside the tunnel in February-March 1980, subsequent to distress of the tunnel; and (3) "WES" borings, emplaced by the Corps of Engineers in April-May 1980. A landfill-trash dump was discovered just east of the tunnel centerline in the WES borings. The natural soils at the site to a depth of approximately 110 ft (elevation 500 ft msl) can be divided into three units. The upper 60 ft is predominantly clay (CL) with varying gravel content, ranging from soft and plastic to very hard and stiff. Occasional sand and silt lenses also occur within the upper 60 ft, but gravelly clay predominates. A distinct well-sorted (poorly-graded), cohesionless sand and silt unit (SP, SM, and ML) persists throughout the length of the section between approximately 60 ft and 90 ft depth (elevation 550-520 msl). The sand and silt unit is difficult to penetrate (high blow counts), but its cohesionless nature gives it a very low apparent strength when removed from the borehole. At the base of the silt-sand unit, a very hard, cohesive, sandy, gravelly clay (CL) to clayey, gravelly sand (SC) is encountered. This unit lies at about the same elevation (approximately 520 ft msl) for the length of the profile. It often is referred to in other boring logs as "hardpan."

a. Landfill dump. The presence of a "sanitary" landfill extending to a maximum depth of about 40 ft was verified in borings WES-2, -3, -5, and -6 (Figure 13). The landfill, believed to have been started in the 1950's and filled in by early 1960, was also noted by the bypass contract foreman at the site, who stated that they encountered loose rubbish with little or no soil when excavating for the temporary bypass east of the tunnel. Boring C-224 (Figure 9), 12 ft west of the centerline, did not encounter the landfill. Several "TB" borings recorded "no sampling" in the upper 40 ft or so of the log and some of them may have indicated the presence of the landfill. However, the WES borings represent the only certain verification of the landfill. The refuse encountered was typical landfill debris with little or no soil content, and included paper products, tin cans, automobile tires, and other rubbish that caved so badly the entire vertical section had to be cased to allow sampling

of the underlying soils. The north-south limits of the landfill are shown on Figure 13 as well as could be determined from borings. Borings WES-7 and -8 did not encounter the fill. The east-west extent of the landfill is not known, but the fill definitely exists just east of the tunnel. The probable significance of the landfill to this investigation is that the excavation replaced 25 to 40 ft of tight clays (low permeability) with loose permeable rubbish that readily fills with water. Such an excavation might also have relieved residual lateral pressures in the soil in the vicinity of Distressed Area 2.

b. Clay strata. The upper 60 ft of gravelly clays is a complex series of glacial lake sediments. WES borings 1 and 4 logged varved soft clays associated with quiet water glacial lake deposition. Much of the gravelly clay and other lean clay (CL) encountered in the upper 60 ft was resistant to penetration by the sampler (standard split spoon and 140-lb hammer) and a few units were considered very hard, indicating a high degree of consolidation. Some samples from the gravelly clay between 20- and 40-ft depth in boring WES-4 were almost shale-like in consistency and hardness. The silts and sands encountered within the upper 60 ft generally all exhibit some cohesion and maintain their shape when cored. An exception is the 5-ft thick sand lens near elevation 555, Sta 124+00 to 129+00, logged as SP, SW, and SM in several borings, that shows only a little cohesion. Blow counts in borings presented in Figure 13 increased abruptly at similar depths (around 50 ft at elevation 560 msl) south of Sta 125+00. The soils at that depth would appear to have been overconsolidated by loading, but because the sediments down to a depth of about 90 ft have been determined to be late Wisconsin glacial lake deposits (para 38), there should have been no thick glacial ice loading of the upper 90 ft. It is also improbable that any great thickness of soil overburden has been subsequently removed in the short geologic time since deposition of the sediments occurred. The glacial till complex below the beach sand unit (Figure 13) is interpreted as older glacial drift and probably has been ice loaded and overconsolidated.

c. Beach sand. The cohesionless well-sorted sand below the gravelly clays is interpreted as a beach ridge developed on the margin of one of the glacial lakes (Mozola, 1969). The importance of the sand is its persistence along the tunnel route near the invert of the tunnel. The unit grades from fine, occasionally

medium-grained sand to very fine (grit just perceptible) to silt size. The silt sizes generally occur toward the base of the unit, as can be seen in borings TB-29, WES-4, TB-27, WES-3, TB-4, and TB-1, Figure 13. The sand is predominantly quartz with varying percentages of light and dark colored accessory minerals of igneous origin. The top of the beach unit, as determined from boring logs, is relatively flat (less than 5-ft relief) along the tunnel line and is exposed along the tunnel above the invert.

d. Distressed Area 1. Detailed 1 to 1 profiles and sections constructed from logs of the closely spaced TB borings made in early February 1980 on either side of the tunnel, in the central portion of Distressed Area 1 are shown in Figure 14. The tunnel profile and sections are for the as-built location before distress. Silty sand, silt, and fine sand exist along the base of the tunnel; clay and silt strata exist above the sand. The material excavated during mining in this segment noted considerable sand along with clay, sandy clay, stones and rocks (Table 7). Borings TB-4 and 8A show loose sand and soft clay strata. Voids were also found adjacent to the invert in TB-5 and TB-12. The sections show the variable stratification on either side of the tunnel. Lines connecting similar strata indicate orientation of silt, clay, and sand around the tunnel periphery, before distress. The density values from split spoon samples are shown on the profiles in Figure 14. Although split spoon samples are considered disturbed samples, the dry densities (excluding the effects of water content) provide a relative measure or index of in situ conditions. Low density values are indicated for the soft to stiff clay above the crown at Sta 132+40 (TB-8 and TB-8A) and in the silt layer at Sta 131+40 (TB-12). Dry densities for other strata range from 113 to 129 pcf. These values are within the range for samples from other locations as shown in Figure 15. It should be noted from Figure 15 that there is a wide range in dry densities at the tunnel site. Thus, low density values may be a result of the natural variation in the complex stratification. Gradation data shown in Figure 14a indicates that the beach sand unit along the base of the tunnel is primarily a silty sand. The sand content increases with depth. Very low blow counts of 2 to 15 (compared to blow counts of 33 to 83 in preconstruction borings C-224 and C-176, Figure 9) can be noted close to the tunnel in Figure 14b. Final collapse of the tunnel crown at Sta 131+80 occurred during the time the TB borings were being drilled

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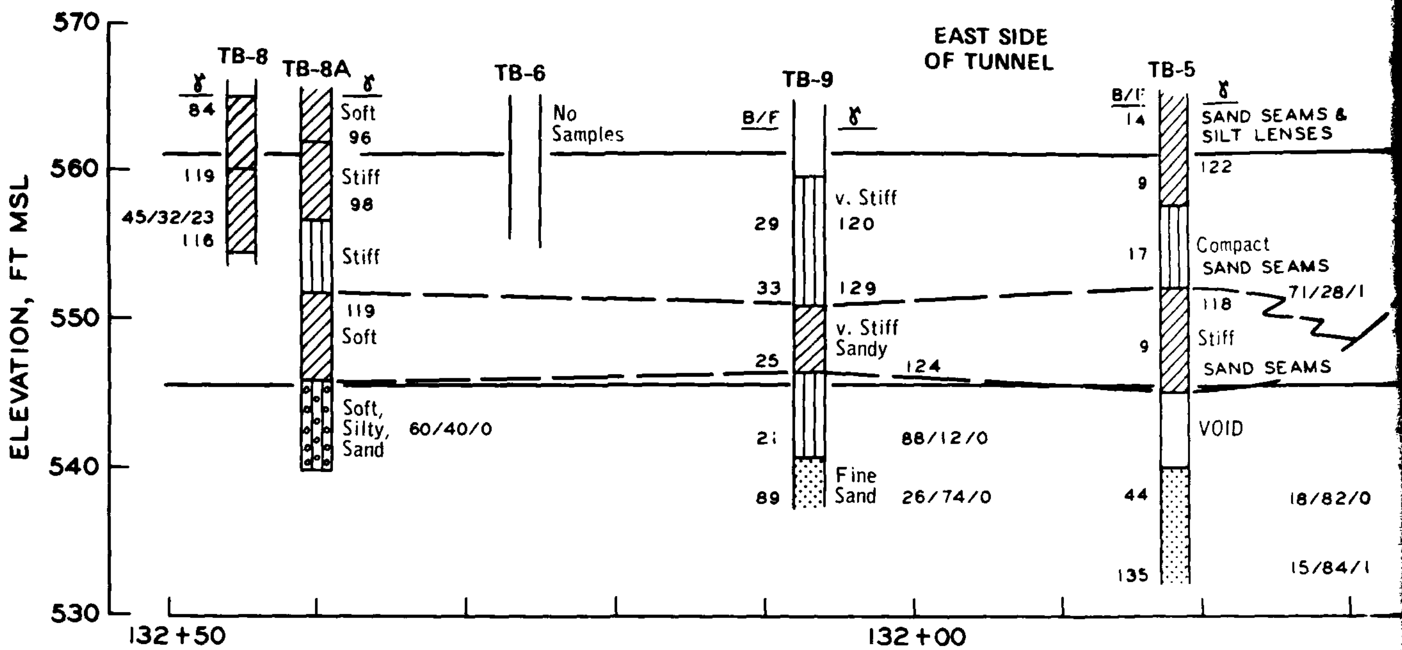
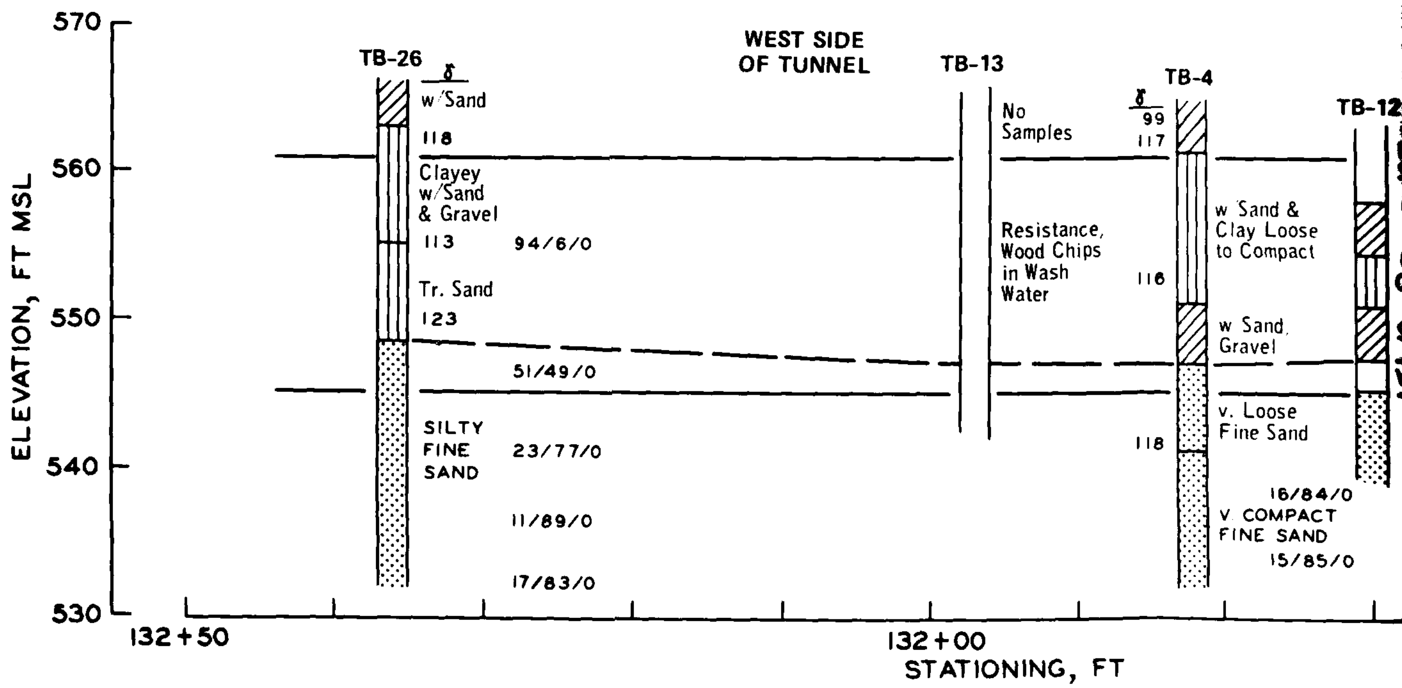
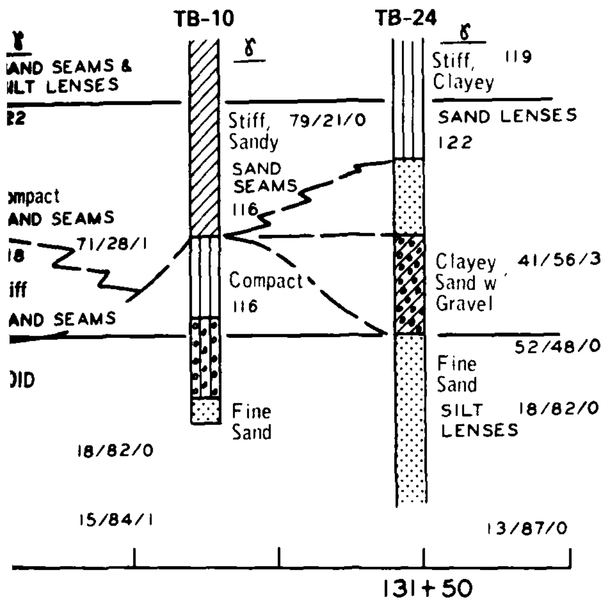


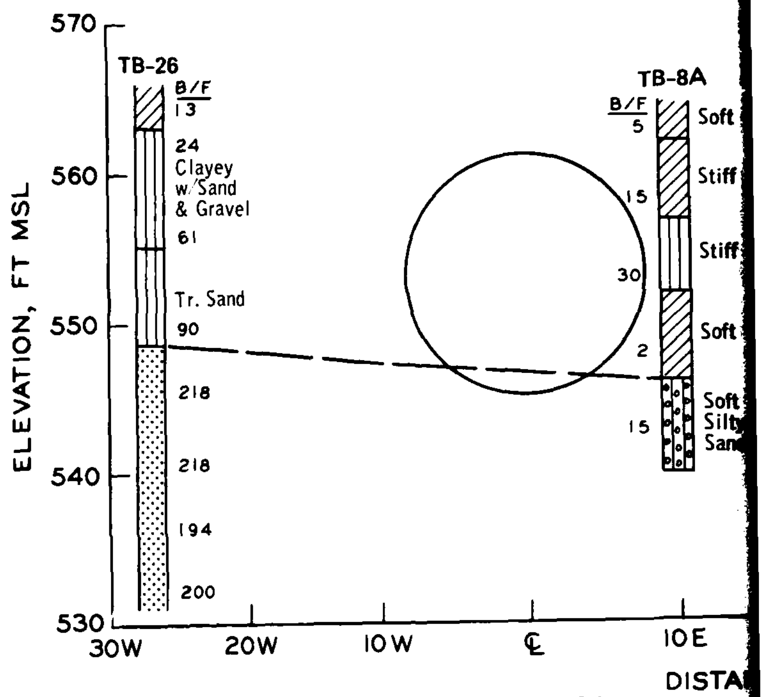
EAST SIDE OF TUNNEL



WEST SIDE OF TUNNEL

a. PROFILES





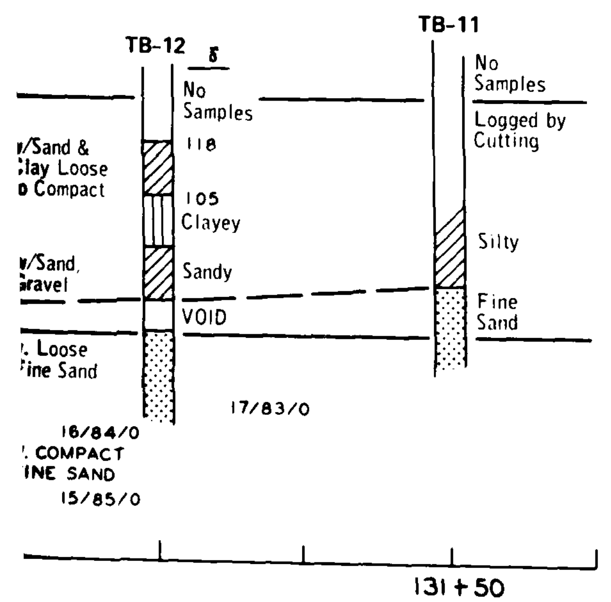
STA 132+38

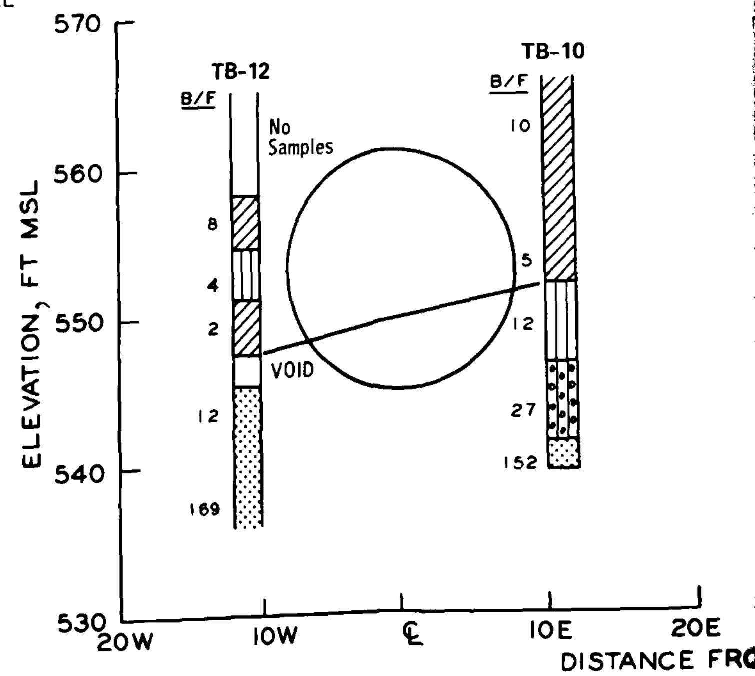
NOTE:

B/F = BLOWS PER FOOT

γ = DRY UNIT WEIGHT, PCF

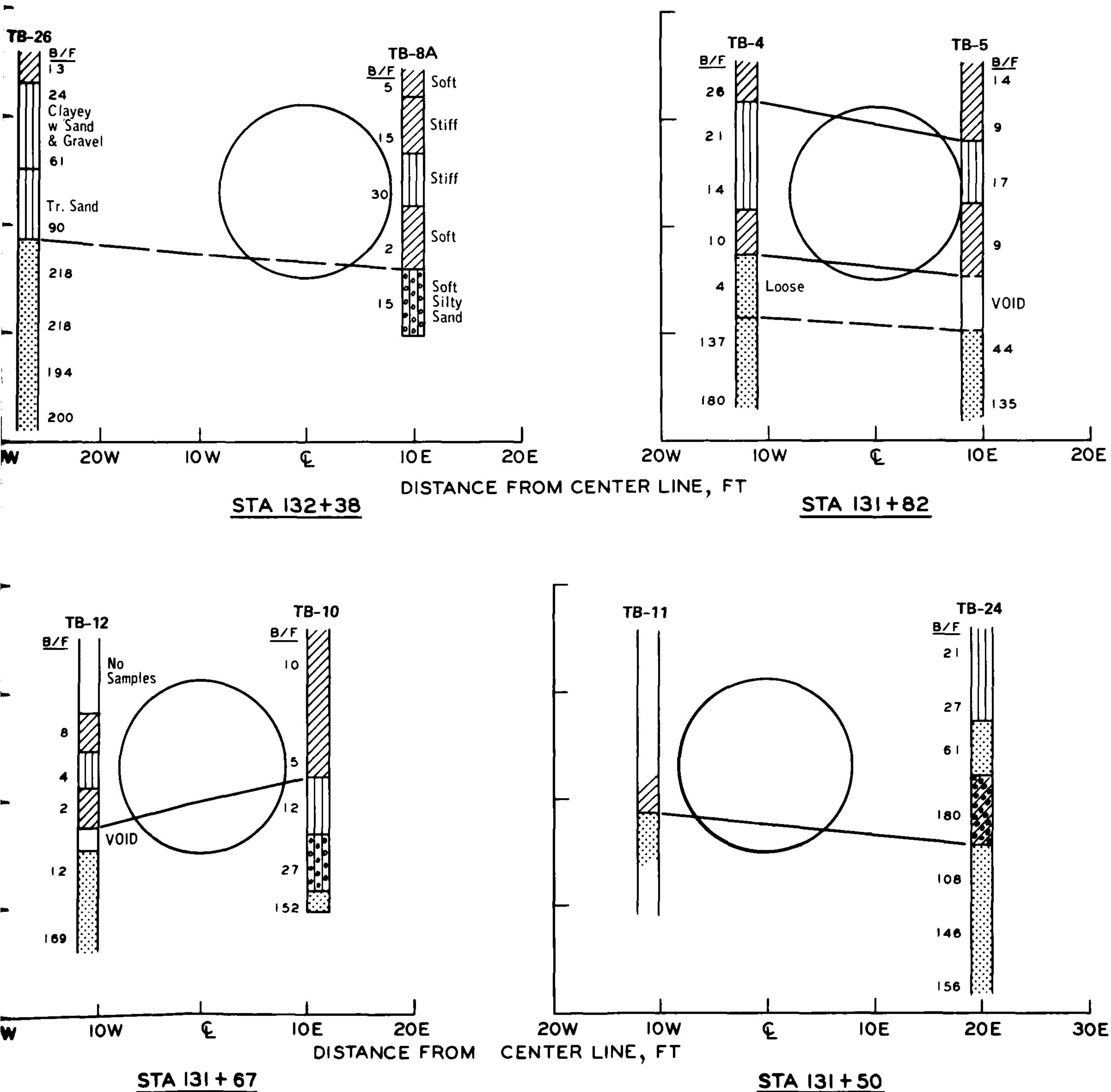
40/55/5 = % SILT OR CLAY/ % SAND/ % GRAVEL





STA 131 + 67

b. S



b. SECTIONS

Figure 14. Profiles and sections in Distressed Area 1, 15 Mile Road/ Edison Corridor Tunnel Failure Study

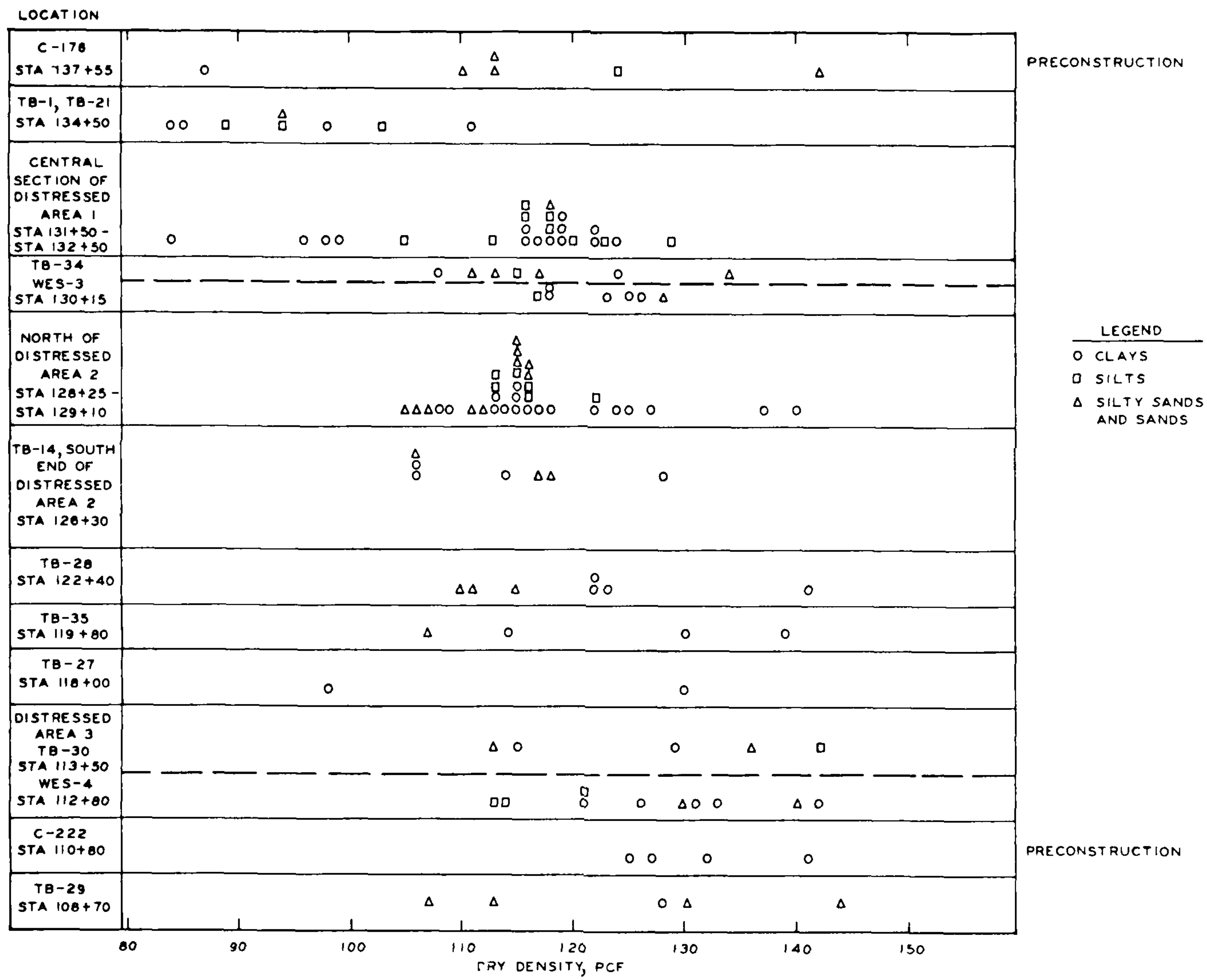


Figure 15. Range of dry densicy values, elevation 535 to 570 ft msl

(site settlement observations, para 61 c). Occurrence of the loose and soft soil conditions and low blow counts prior to collapse or as a result of collapse cannot be determined. Additional sections of soil stratification just north and south of Distressed Area 1 are shown in Figure 16. Although considerable variation in stratification persists in the east-west direction, TB-1 north of TB-26 (Figure 14a) shows the same silt-sand sequence with silt deepening to the north. Strata changes between TB-24 and WES-3 are shown directly in Figure 13. At Sta 134+50 (Figure 16), low dry densities and low blow counts were measured 17 ft from the tunnel sides in the silt to the west (TB-1) and silt and clayey sand to the east (TB-21), indicating loosening of these materials north of the distress zone. Figure 15 shows a lower range of dry densities in comparison to other locations. At Sta 130+15, south of Distressed Area 1 (Figure 16), dry densities and blow counts were generally higher than those at Sta 134+50 and in Distressed Area 1. The beach sand unit at Sta 130+15 has considerable sandy silt (TB-34). The section of Sta 119+90 in Figure 16 is referred to later.

e. Nondistressed section between Distressed Areas 1 and 2. Between Distressed Areas 1 and 2, the beach sand unit grades from interbedded silt and fine sand to fine sand along the tunnel invert and then dips to elevation 531 (Figure 13, borings WES-3 to TB-17) some 5 ft below the tunnel invert, at the north end of Distressed Area 2. Clay and silt strata occur above the beach sand. The 1 to 1 profiles and sections for closely spaced TB borings (extending 72 ft north of Distressed Area 2) in Figure 17 show gradational but relatively continuous clays, silt, and sand strata, respectively, above the beach sand. The gradation data for strata shown as sand and silty sand in Figure 17 indicates considerable sandy silt lenses. The permeability of three sandy silt samples from TB borings (Figure 17) was 10^{-6} cm/sec and 10^{-5} cm/sec for one sample of silty sand. During excavation (as the tunneling progressed south), soils were noted as hard sandy clay to hard clay with sandy spots, stones and rock. The material changed at Sta 128+76 to sand with traces of clay and pebbles to large stones (Table 7). As shown in Figure 17, some samples of clay above the crown and of sand at the base of the tunnel had low dry densities of 105 to 109 pcf (TB-3, TB-22, and TB-17). Other dry densities in the clay, silt, and sand strata ranged from 111 to 118 pcf and from 122 to 140 pcf in the stiff to

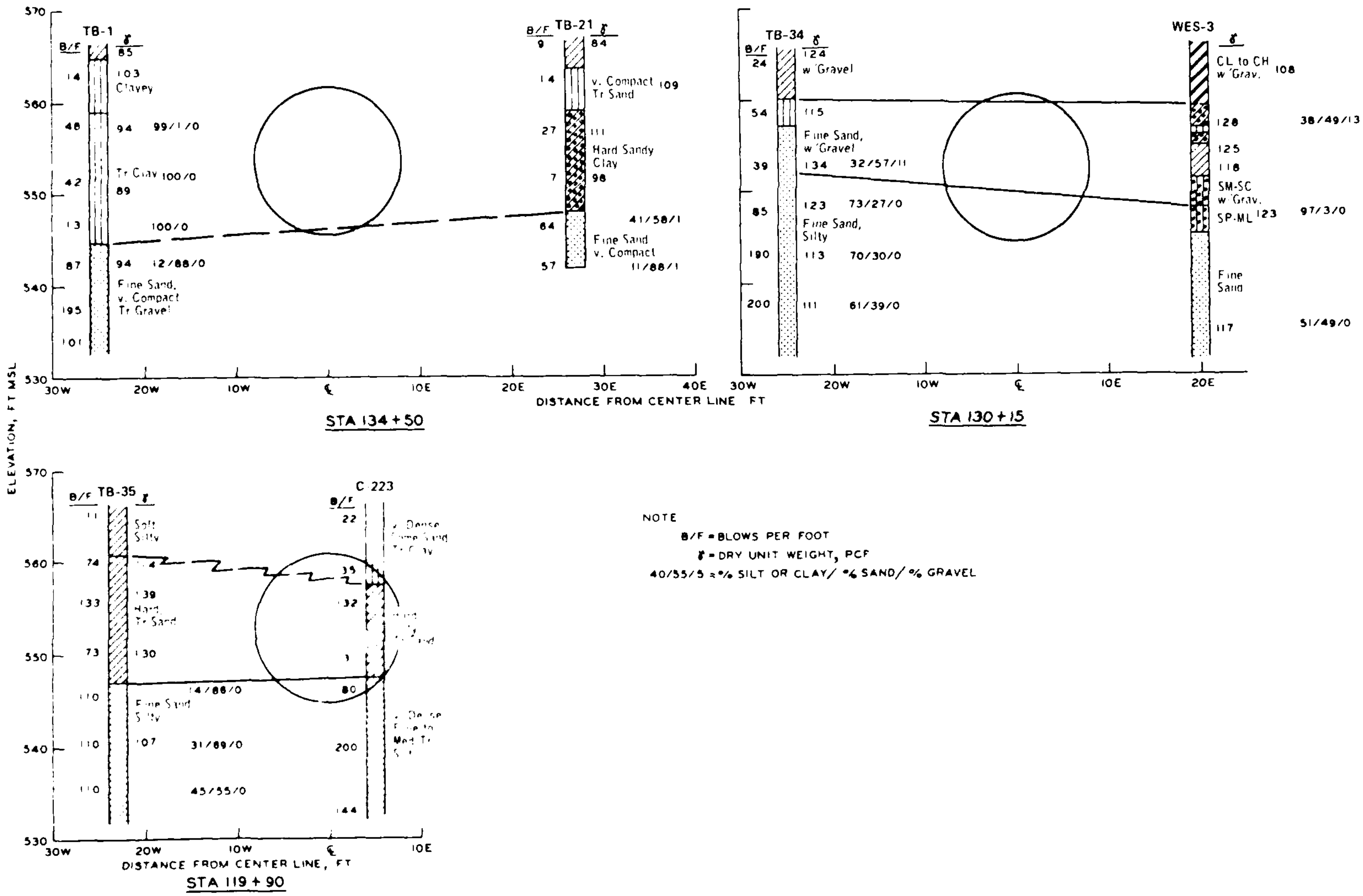


Figure 16. Sections north and south of Distressed Area 1 and south of Distressed Area 2

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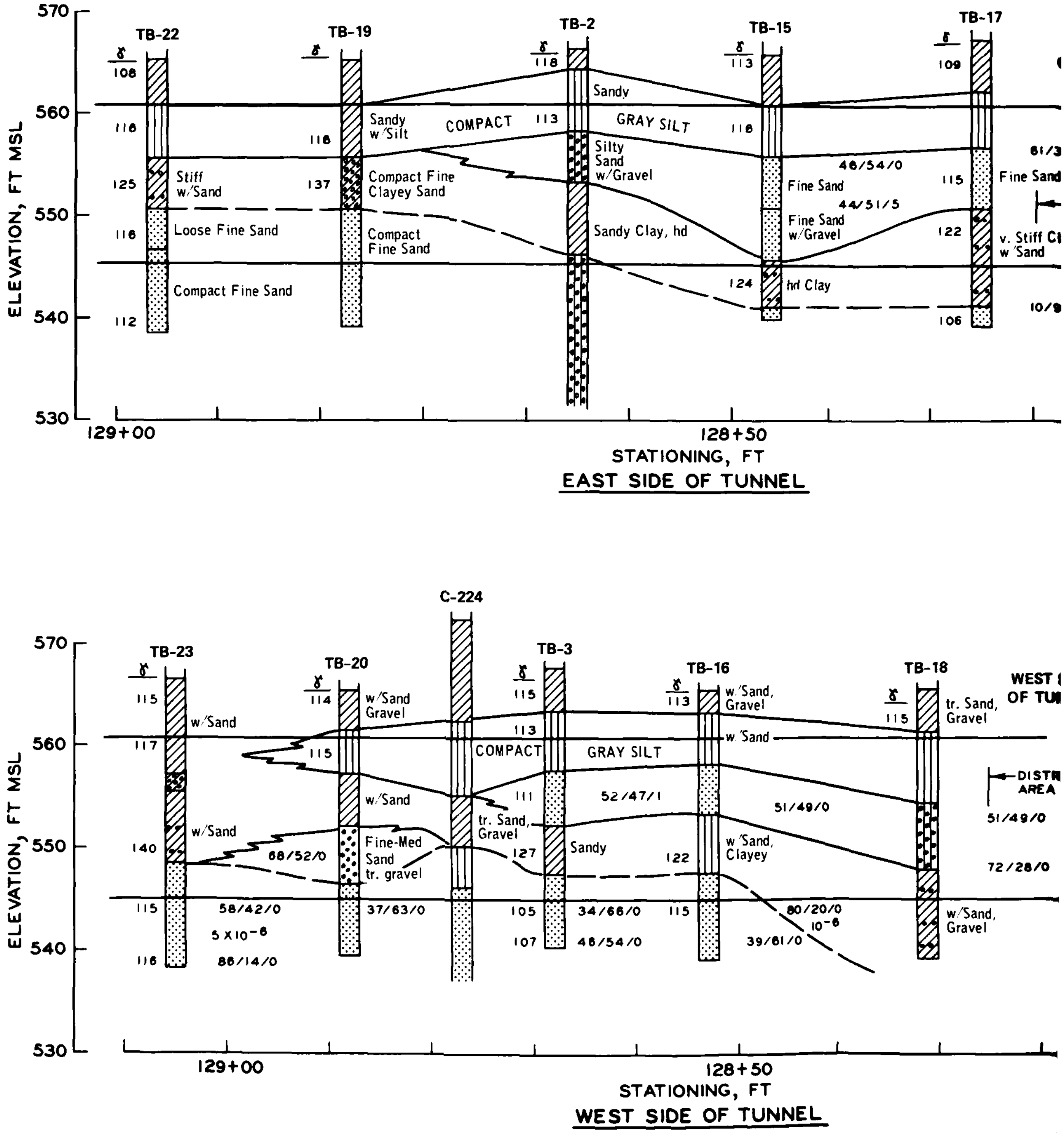
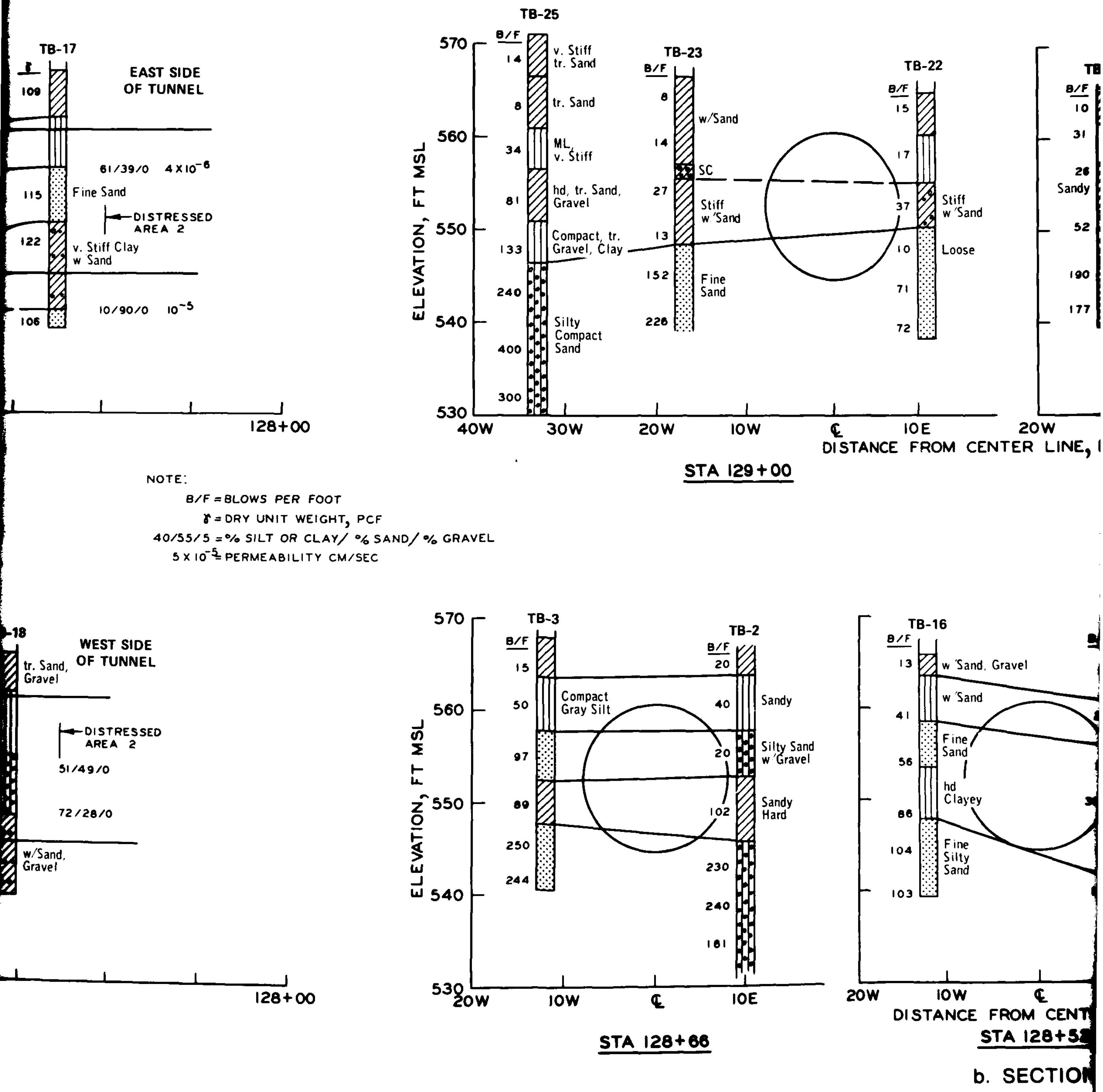
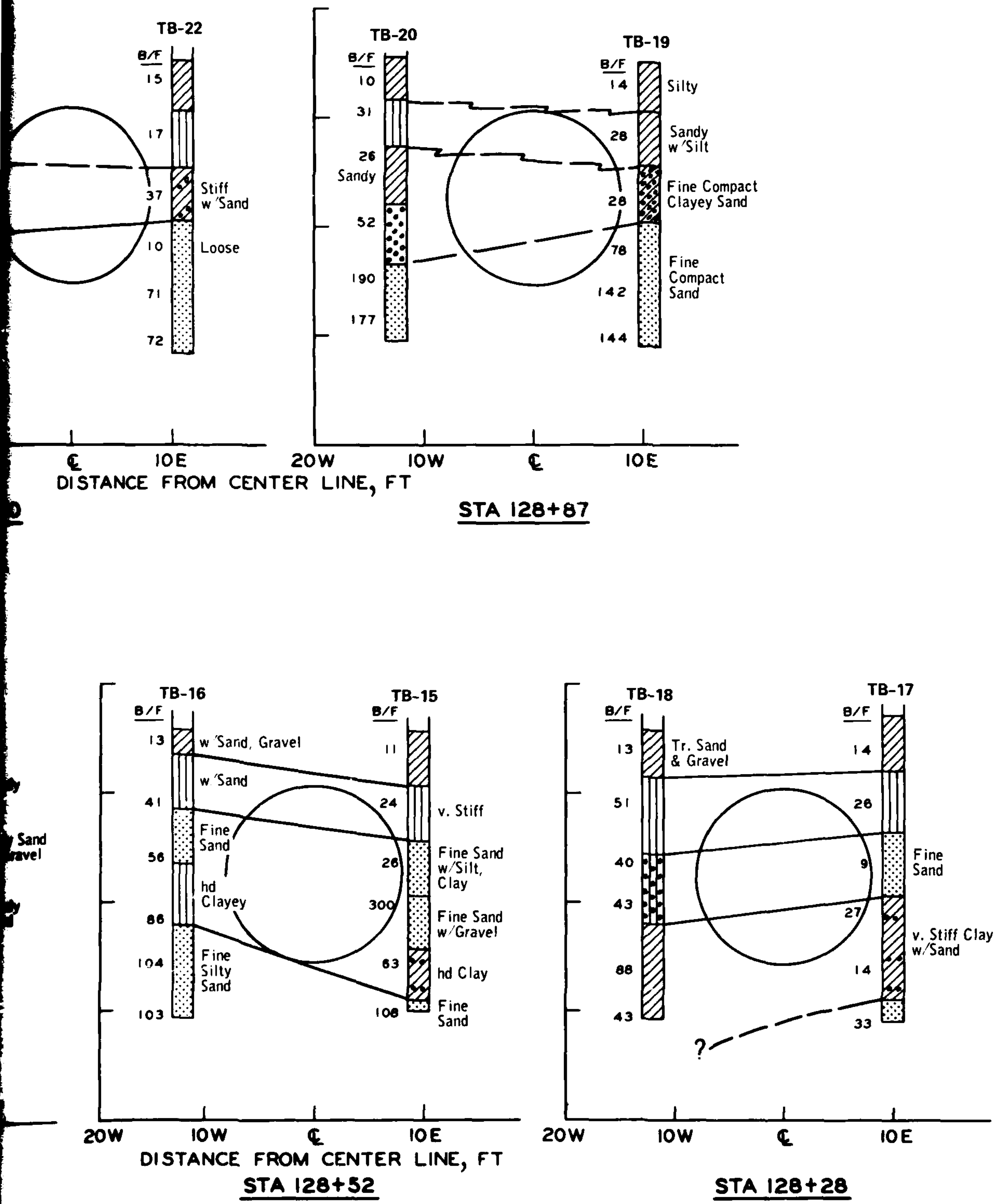

ELEVATION, FT MSL
570
560
550
540
530
TB-22
TB-19
TB-2
TB-15
TB-17
Sandy
COMPACT
GRAY SILT
Sandy w/Silt
Silty Sand w/Gravel
Stiff w/Sand
Compact Fine Clayey Sand
Fine Sand
Loose Fine Sand
Compact Fine Sand
Sandy Clay, hd
Fine Sand w/Gravel
v. Stiff Cl w/Sand
hd Clay
Compact Fine Sand
46/54/0
44/51/5
129+00
128+50
STATIONING, FT
EAST SIDE OF TUNNEL
C-224
TB-23
TB-20
TB-3
TB-16
TB-18
w/Sand
w/Sand Gravel
w/Sand, Gravel
tr. Sand, Gravel
COMPACT
GRAY SILT
w/Sand
tr. Sand, Gravel
Fine-Med Sand tr. gravel
Sandy
w/Sand, Clayey
w/Sand, Gravel
52/47/1
51/49/0
68/52/0
58/42/0
37/63/0
34/66/0
80/20/0
46/54/0
39/61/0
88/14/0
72/28/0
5 X 10^{-6}
10^{-6}
DISTR AREA
WEST OF TU
129+00
128+50
STATIONING, FT
WEST SIDE OF TUNNEL

a. PROFILES


TB-17
EAST SIDE OF TUNNEL
109
115
122
106
61/39/0 4 X 10^-6
Fine Sand
DISTRESSED AREA 2
v. Stiff Clay w Sand
10/90/0 10^-5
128+00
TB-25
TB-23
TB-22
B/F
570
560
550
540
530
ELEVATION, FT MSL
14
8
34
81
133
240
400
300
v. Stiff tr. Sand
tr. Sand
ML, v. Stiff
hd, tr. Sand, Gravel
Compact, tr. Gravel, Clay
Silty Compact Sand
8
14
27
13
152
226
w/Sand
SC
Stiff w Sand
Fine Sand
15
17
37
10
71
72
Stiff w Sand
Loose
10
31
26
Sandy
52
190
177
40W
30W
20W
10W
℄
10E
20W
DISTANCE FROM CENTER LINE,
STA 129+00
NOTE:
B/F = BLOWS PER FOOT
γ = DRY UNIT WEIGHT, PCF
40/55/5 = % SILT OR CLAY/ % SAND/ % GRAVEL
5 X 10^-5 = PERMEABILITY CM/SEC
-18
WEST SIDE OF TUNNEL
tr. Sand, Gravel
DISTRESSED AREA 2
51/49/0
72/28/0
w/Sand, Gravel
128+00
TB-3
TB-2
TB-16
15
50
97
89
250
244
Compact Gray Silt
20
40
20
102
230
240
161
Sandy
Silty Sand w Gravel
Sandy Hard
13
41
56
86
104
103
w Sand, Gravel
w Sand
Fine Sand
hd Clayey
Fine Silty Sand
20W
10W
℄
10E
STA 128+66
DISTANCE FROM CENT
STA 128+5
b. SECTIO



b. SECTIONS

Figure 17. Profiles and sections along nondistressed tunnel segment between Distressed Areas 1 and 2, 15 Mile Road/Edison Corridor Tunnel Failure Study

hard sandy clay to clayey silt stratum dipping from the springline at Sta 129 to the tunnel base at Sta 128+50. These dry density values are compared with those at other locations in Figure 15. Only borings TB-22 and TB-17 encountered a loose sand interval with low blow counts of 9 and 10. Low blow counts of 8 to 20 in clays, just above the tunnel are similar to those in preconstruction borings C-224 and C-176 (Figure 9). High blow counts adjacent to the tunnel are also comparable to those for the preconstruction borings. The soil conditions along this undistressed segment are similar to conditions before construction.

f. Distressed Area 2. In Distressed Area 2 the fine-grained beach sand rises back above the tunnel invert and a stratum of fine to very fine sand persists along the tunnel between the springline and crown (Figure 13). A compact to soft silt layer exists along the tunnel crown. Tunnel excavation was noted as (1) sand mixed with traces of clay, pebbles, and large stones from 128+76 to 127+76, (2) clay, very rocky to 127+56, (3) clay to sandy clay with boulders, rocks, and scattered stones to 125+81, and (4) hard clay, sand, traces of soft clay in crown, some water appearing, also scattered stones (Table 7). Test boring TB-14 at the south end of Distressed Area 2 showed low dry densities of 106 pcf and low blow counts of 11 and 12 in the silty clay near the tunnel crown.

g. Nondistressed section between Distressed Areas 2 and 3. Between Distressed Areas 2 and 3, the beach sand continues at a slightly decreasing level along the tunnel base as shown in Figure 13. A relatively thick strata of hard to very hard silty to sandy clay with variable gravel content occur along the tunnel above the sand with intermittent silt strata above the clay. High blow counts in boring WES-2 and the TB borings are similar to those before construction. Soil conditions indicated by these borings are considered representative of those adjacent to the tunnel. The section at Sta 119+50 (Figure 16) shows correlation of strata in the east-west direction between borings TB-35 and the preconstruction boring C-223 and blow counts that are similar. During tunneling along this segment, sand was noted only from 123+46 south to 122+61 (Table 7); sandy clay with rocks and stones was the general soil condition noted.

h. Distressed Area 3. Along Distressed Area 3, the fine-grained beach sand dips below the tunnel invert (Figure 13), but strata of sand, sand seams, and clayey

sand with much gravel (boring WES-4) are prevalent. A detailed section along this segment shown in Figure 18 indicates the sand and silt layers and prevalent sand seams. Both the as-built and after distress tunnel profiles are shown. Boring WES-9 and WES-10 adjacent to the tunnel sides lost considerable drilling fluid down to elevations 570 and 563. Drilling fluid from WES-9 was found entering the tunnel the day following completion of the boring. Very low blow counts were indicative of the soft to very soft soil conditions found in boring WES-9. Boring WES-10 indicated relatively low blow counts to a depth of 8 ft below the tunnel. The higher blow counts in boring TB-30 can be explained by the fact that this boring was 24 ft further west of the tunnel than boring WES-10, and soils at TB-30 were probably less affected by the tunnel distress. Both borings TB-30 and WES-4 indicated relatively high dry densities (Figure 15) which compare with those for samples from the preconstruction boring C-222. As discussed subsequently (para 61 g), the sand layers and seams are water bearing. Notes on tunnel excavation materials along this segment included "sand on bottom" (115+41 to 115+11), "sand seams in bottom" (114+66 to 114+36), "sand layer in face at springline" (113+71 to 113+31), "sandy bottom and top" (113+31 to 111) and traces of sand indicative of sand lenses from Stas 114+86 to 114+66 and 113+11 to 112+76. Seepage water was also noted from 113+11 to 112+76.

i. South of Distressed Area 3. South of Distressed Area 3, the beach sand dips below elevation 540. A thick stratum of well-graded sand and gravel with boulders overlain by sandy silts and silty sands occurs along the tunnel at boring TB-29 and Sta 108+70 where caving ground problems started during mining. Samples from boring TB-29 indicated a wide range of dry densities (Figure 15).

61. Conditions during site stabilization measures. Tunnel conditions during grouting, freezing, and dewatering were modified as well as the soil conditions around the tunnel. While these measures were necessary, they undoubtedly modified in situ stresses and soil properties. The sequence of stabilization measures, conditions occurring during their implementation and effects on in situ stresses and soil properties are presented in the following section.

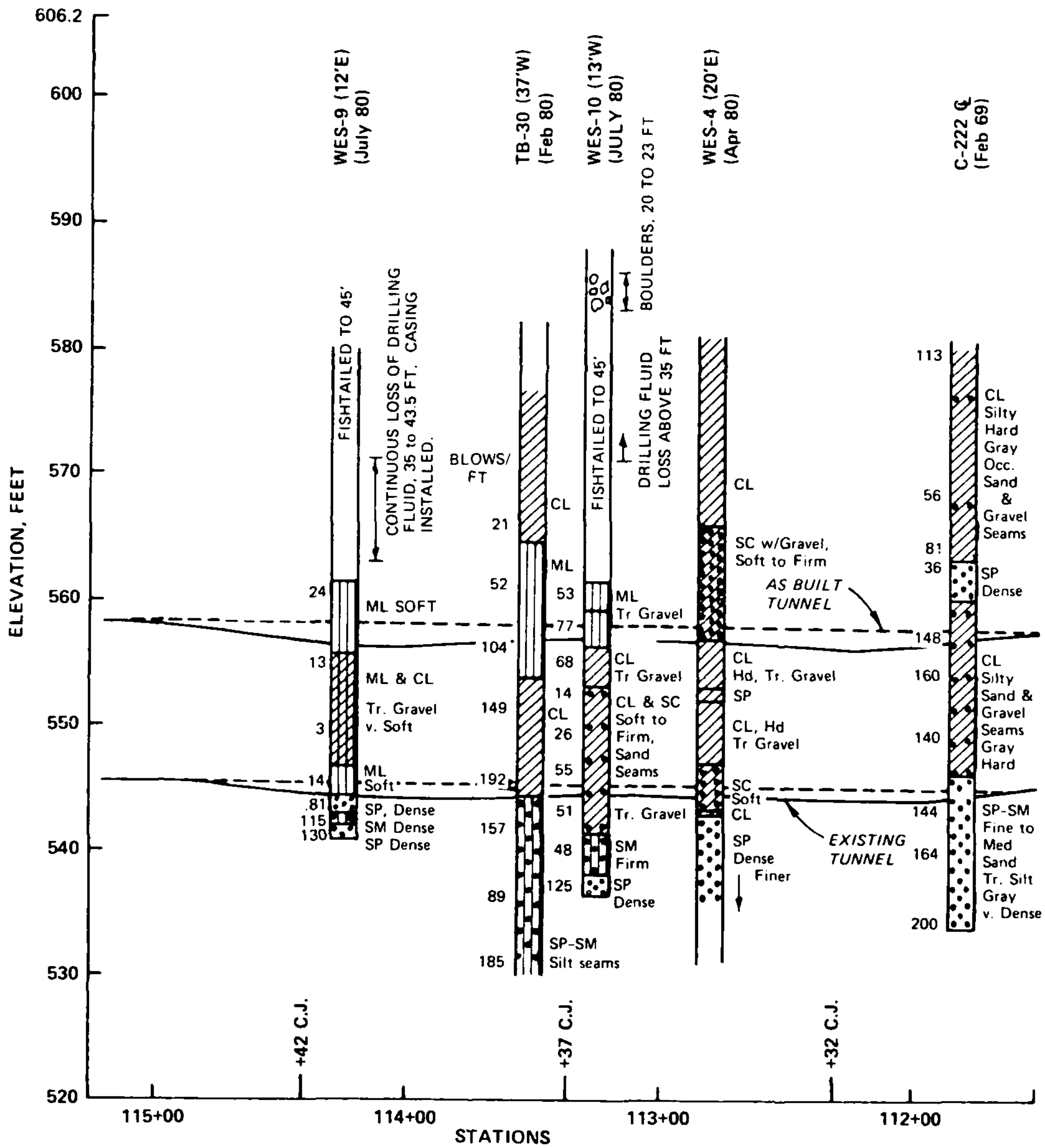


Figure 18. Stratification and soil conditions, Distressed Area 3

a. Sequence of operations. Closely spaced TB borings in Distressed Area 1 and just north of Distressed Area 2 were started 1 February 1980 and continued through 28 February 1980. Piezometers were installed in selected TB borings and grouting was done in other TB borings. Grouting operations, summarized in Table 8, were accomplished from 2 February 1980 to 7 February 1980. Borings ("SP")made to install settlement pins were started 2 February 1980 and completed on 13 February 1980. The tunnel was still in operation. Dewatering at well No. 1 started on 18 February 1980 and all wells were in operation by about 10 March 1980.

b. Tunnel crown collapse, Distressed Area 1. In Distressed Area 1, some TB borings and grouting were completed before and others after the first settlement pin readings as summarized in Table 9. A summary of settlement pin data is shown for Distressed Areas 1 and 2 in Figure 19. The as-built and deformed tunnel profiles are shown in the figure for correlation with settlement pin data. When installed, pins 5 through 9 showed that the crown had sagged about 5 ft. Pins 5 and 9 indicated further movement totaling 1 to 3 ft from 5 February 1980 to 20 February 1980 (plot of elevation versus date in Figure 19). Table 9 shows that significant grout was injected in TB-4, TB-12, TB-5, and TB-10 on 2 February 1980 through 5 February 1980 before settlement readings. After initial pin readings, grout was injected in TB-11, TB-8A, and TB-6 on 6 February 1980 with 50 cu ft taken in TB-11 and 150 cu ft in TB-6. After completion of the TB borings on 8 February 1980 and grouting on 6 February 1980, pins 5 and 9 showed an additional downward movement of the crown. Based on this sequence of events, it is possible that further fall-in of the crown and worsening of distress could have been due, in part, to the boring and grouting operations.

c. Surface settlement, Distressed Area 1. Because of site operations, the ground surface was being modified and apparently no record was made of any surface depressions over break areas. Using ground surface elevations recorded on TB and SP boring logs, estimated contours of the ground surface over break area 1 as of 8 February 1980 was developed and is shown in Figure 19. An apparent surface depression of about 1.2 ft was indicated. This depression could have resulted from collapse at break 1 and settlement or deforming of clay strata and dump material above the break.

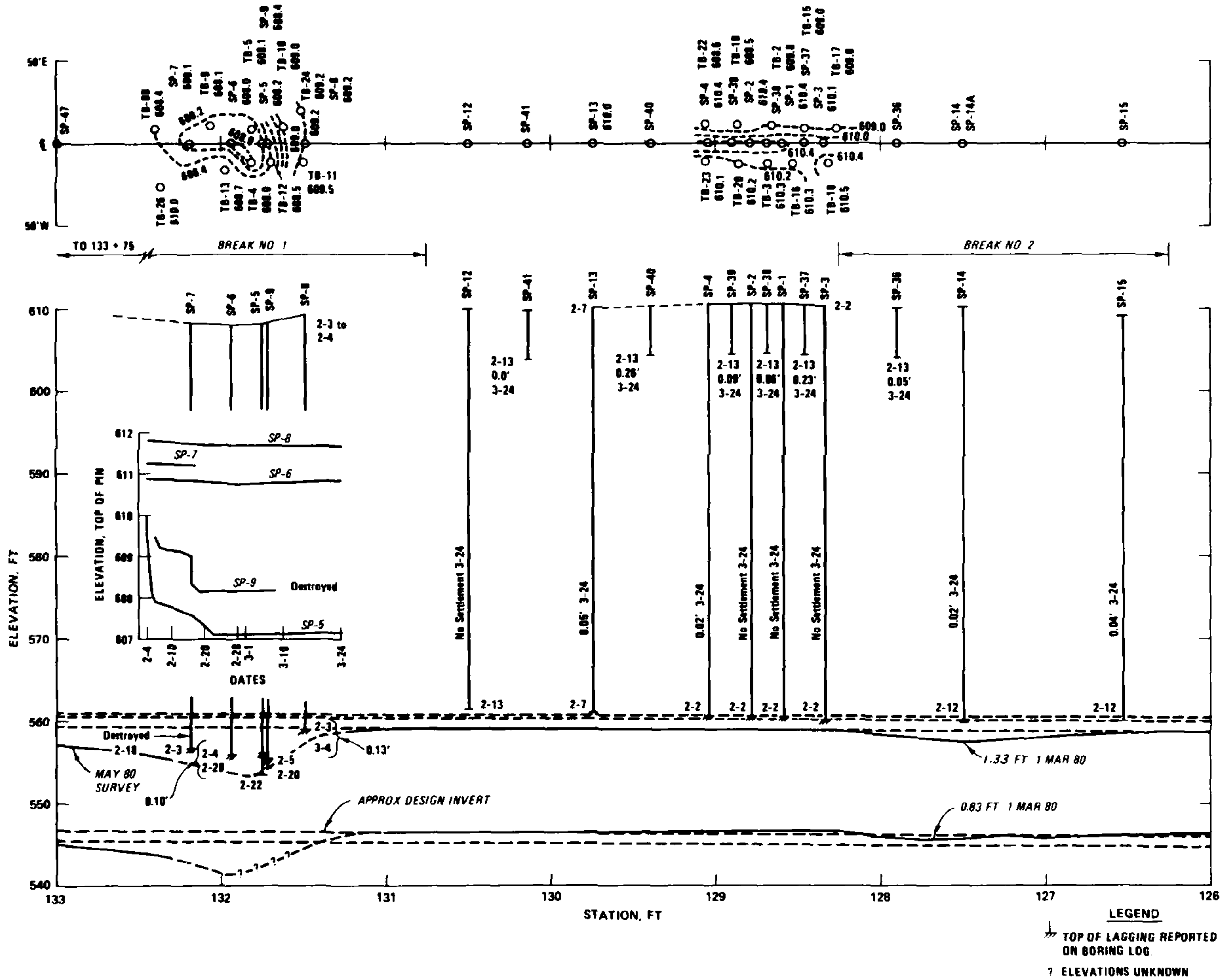


Figure 19. Settlement pin observations, Distressed Areas 1 and 2, 15 Mile Road/Edison Corridor Tunnel Failure Study

d. Ground freezing, Distressed Area 1. Ground freezing of the collapsed section in Distressed Area 1 apparently started on 14 February 1980 with row 9. All freeze pipes were apparently in operation by 29 February 1980. The ground freezing prevented further movement and deformation around this section and thus helped preserve in situ and tunnel collapse conditions and any caving above the tunnel crown.

e. Distressed Area 2 to 3. Just north of Distressed Area 2, closely spaced borings and grouting were accomplished after initial settlement pin readings. In the five TB borings where grout was injected, grout take was recorded only at TB-17 (27 cu ft) at the north end of Distressed Area 2 where 5 ft of fine sand existed at the east springline with a low blow count of 9 (Figure 17). Settlement pin readings in this area indicated only a small settlement of 0.02 ft at SP-4. SP-14 in the middle of area 2, but apparently not seated on the wood lagging, showed only 0.02 ft settlement through 24 March 1980 while visual measurement in the tunnel showed a crown sag of 1.33 ft and 0.83 ft settlement of the invert on 1 March 1980. Thus, the tips apparently were founded in silt above a void created when the tunnel crown moved downward. Settlements of other deep pins south of Distressed Area 1 and through Distressed Area 2 showed either no settlement or settlements of 0.02 to 0.05 ft (0.6-in. or less) and their tip elevation except for SP-14 correlate with the existing top of tunnel. Shallow pins indicated random settlements as high as 0.26 ft (3.5 in.), which could have resulted from surface operations (operating equipment destroyed several pins) or by freezing weather and ice recorded on data sheets of pin readings.

f. Settlement of Distressed Area 3. The settlement pin data for area 3 is summarized in Figure 20. The tip elevations of the pins indicated significant crown displacement had taken place before pins were installed on 6 February 1980. The deep pins recorded as being in contact with the lagging (SP-19 and 20) showed no further crown settlement as of 24 March 1980. SP-18 founded at the same depth as SP-20 also showed no settlement, while SP-17A indicated 0.02 ft on 4 March 1980. The tip elevations of the pins agree closely with the estimated existing top of tunnel lagging. Shallow settlement pins, along the tunnel centerline, indicated settlement of 0.11 ft to 0.60 ft which tend to correlate with the variation in tunnel crown movement. Pins SP-25 and SP-24, offset 38 ft west of centerline indicated no

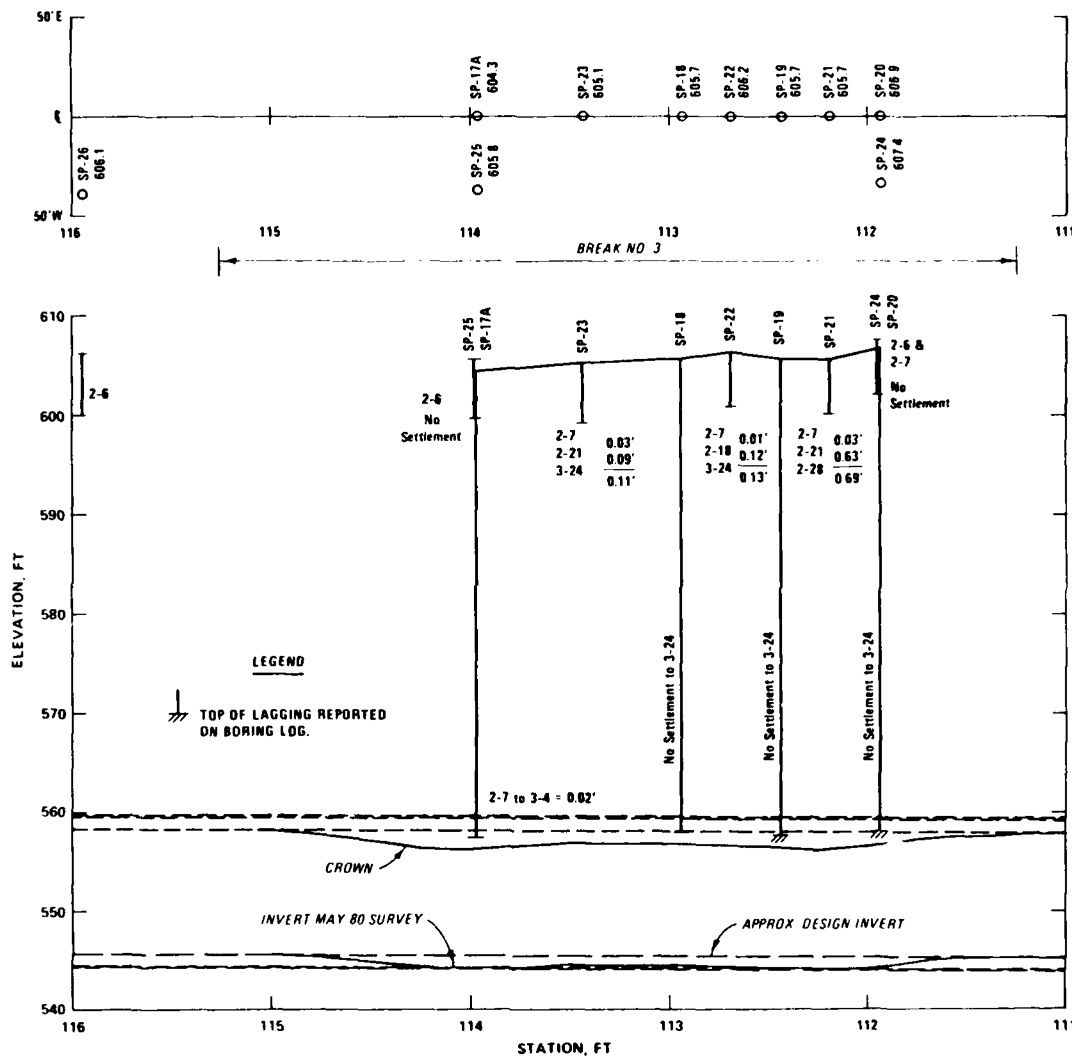


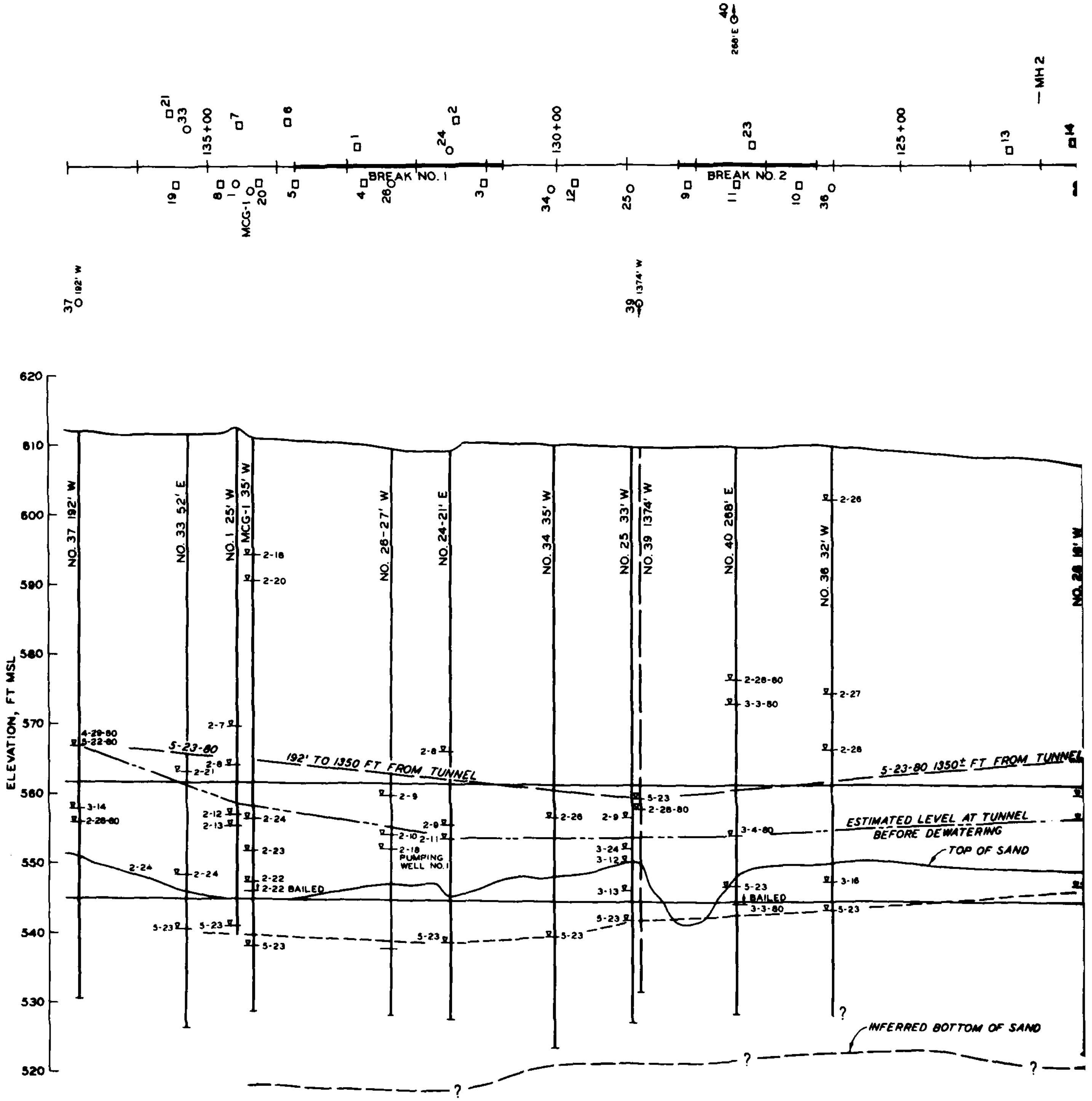
Figure 20. Settlement observations, Distressed Area 3, 15 Mile Road/Edison Corridor Tunnel Failure Study

settlement. It is possible that a narrow surface depression formed as a result of tunnel settlement.

g. Site dewatering and groundwater conditions. Dewatering at the site reduced water pressures around the tunnel and increased vertical ground pressures to the wet weight of the overburden. Dewatering could, thus, have caused additional settlement. Operation of dewatering wells generally started at Distressed Area 1 on 18 February 1980 and progressed southward with all wells pumping by 10 March 1980. Since the surface settlement over break 1 was deduced from ground surface elevations at TB and SP borings completed on 8 March 1980 prior to dewatering, the surface settlement was not affected by dewatering. However, shallow settlement measurements after 18 February 1980 probably were affected by dewatering especially north of Distressed Area 2 and over Distressed Area 3. On the other hand, piezometer data, recorded to monitor dewatering success, provided valuable information in groundwater at the site.

(1) Locations. A plan map of piezometer and dewatering well locations and a profile of recorded water levels in piezometers is shown in Figure 21. Piezometer locations were mainly along the tunnel. However, two piezometers (Nos. 39 and 42) were installed 1300 to 1400 ft west of the tunnel and three piezomters were located 268 ft (No. 40) and some 1300 ft (Nos. 43 and 44) east. One additional piezometer (No. 37) was located 192 ft west at Sta 136+88. All piezometer tips were set into the beach sand unit to a depth of ±85 ft with tips at elevations of about 530 to 515 depending on ground surface elevations.

(2) Installation and reading. Most of the 8-in.-diam piezometer borings (MCG-1, TB-1, TB-24 to TB-29, TB-33, TB-37 to TB-39, TB-41 to TB-43) were drilled by rotary drilling methods using water. Borings TB-37, TB-39 and TB-41 were drilled using hollow stem augers (no drilling water) and recorded groundwater depths were not affected by drilling. Water was encountered in TB-37 at elevations 576 and 550. The elevation of 576 is 5 ft lower than that found in 1969 in Boring C-176 and elevation 550 is 5 ft higher than that found in drilling the 1970 preconstruction well near the same location. Frequent water level readings were taken and usually stabilized in a few days. Selected piezometers were bailed and recovery readings were taken to verify functioning of the piezometer. Initial water levels and selected levels by date are shown in Figure 21. Bailed


135+00
130+00
125+00
BREAK NO. 1
BREAK NO. 2
MCG-1
MH 2
ELEVATION, FT MSL
NO. 37 192' W
NO. 33 52' E
NO. 1 25' W
MCG-1 35' W
NO. 28-27' W
NO. 24-21' E
NO. 34 35' W
NO. 25 33' W
NO. 39 1374' W
NO. 40 288' E
NO. 36 32' W
5-23-80
192' TO 1350 FT FROM TUNNEL
5-23-80 1350± FT FROM TUNNEL
ESTIMATED LEVEL AT TUNNEL BEFORE DEWATERING
TOP OF SAND
INFERRED BOTTOM OF SAND
PUMPING WELL NO. 1
2-22 BAILED
BAILED

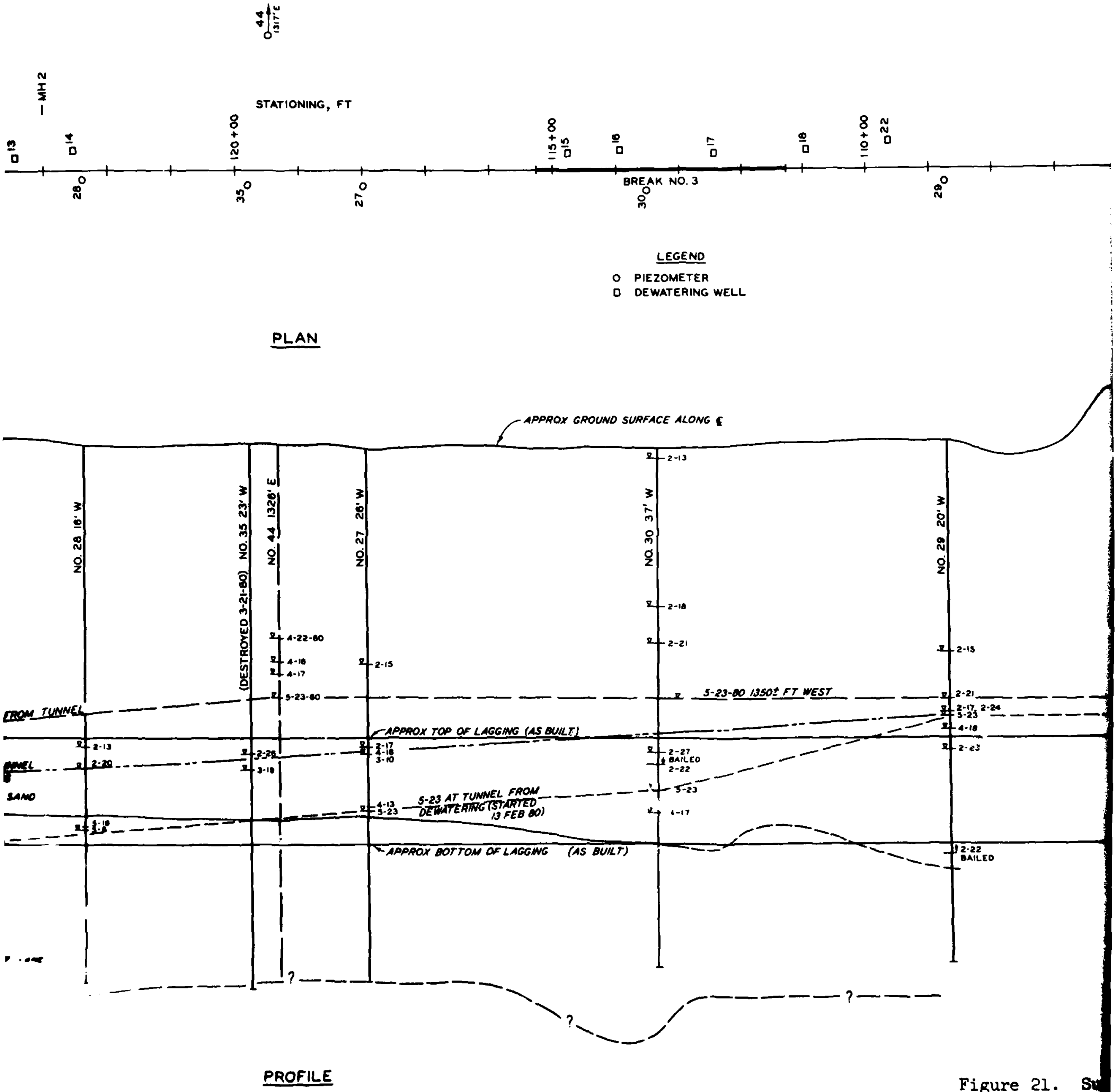


Figure 21. S
15 Mile Road/

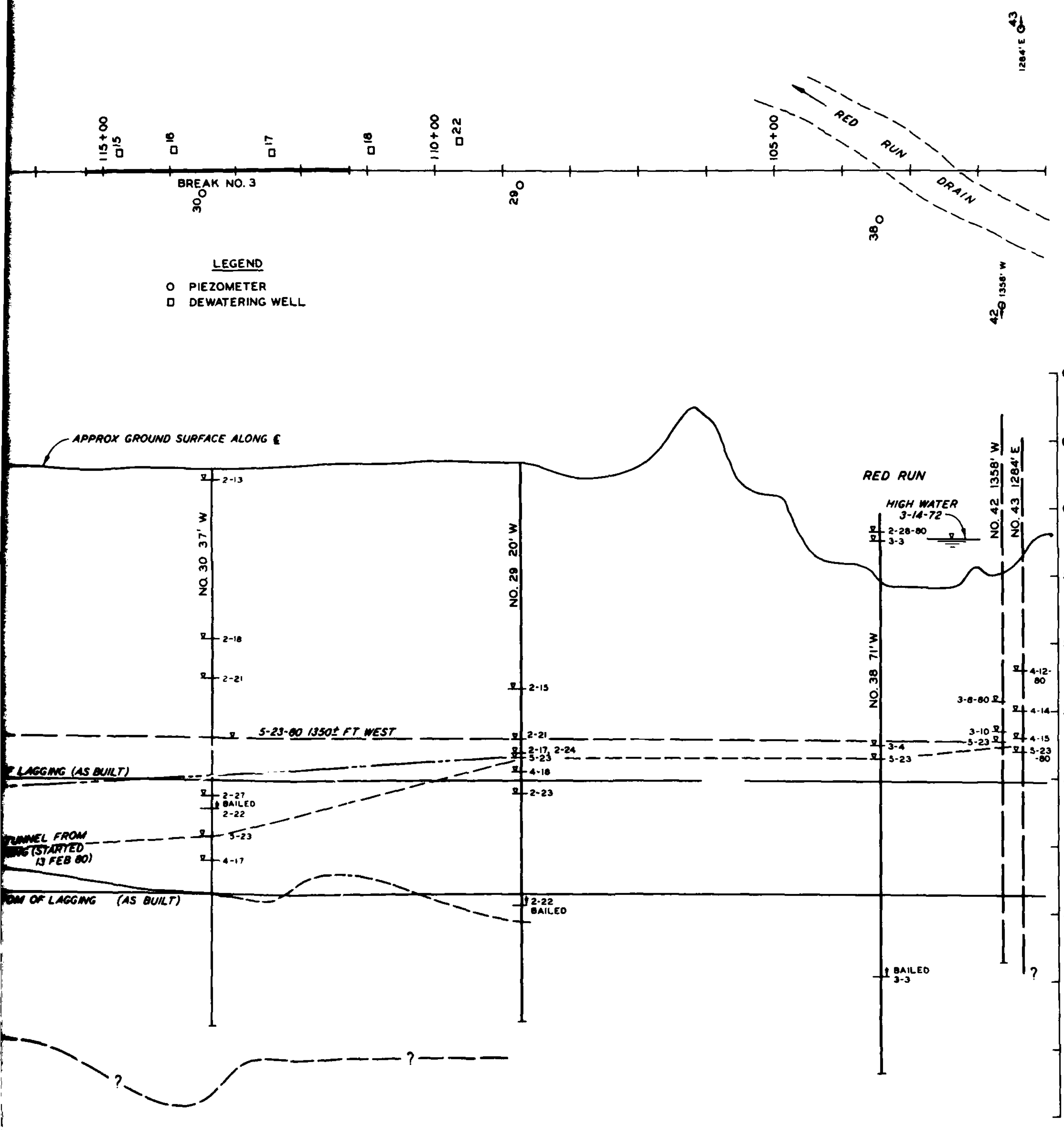


Figure 21. Summary of groundwater conditions, 15 Mile Road/Edison Corridor Tunnel Failure Study

3

piezometers recovered within one to two days indicated relatively high mass permeability of the sand unit.

(3) Groundwater levels. Three water profiles are shown on Figure 21. The upper profile is for conditions 192 to 1400 ft away from the tunnel after stabilization of piezometer levels in the sand which were not significantly influenced by dewatering. The second profile is for piezometers near the tunnel before the effects of dewatering. The third and lowest profile is along the tunnel after dewatering had its maximum effect. The two lower profiles slope upward to the south where they tie into the profile uneffected by dewatering. This trend indicates increased stratification of clays and water bearing sands to the south, as indicated by borings in Distressed Area 3 (Figure 18). The trend is supported by the fact that the five wells from Sta 115 south could not pull the water level as far down as wells north of Sta 122 (Figure 21). The water profiles before dewatering along the tunnel and at a distance of 1300 ft to 1400 ft from the tunnel dip down in the vicinity of Distressed Area 2. The reason is unclear but could be caused by a southwesterly seepage trend since the beach sand appears to slope to the southeast (based on the top of beach sand at elevations 560 in boring A-34 on Schoenherr Rd east of the site and elevation 530 in boring C-26 on Maple Rd to the west). Also the dip in the top of the beach sand at Sta 128 could indicate a buried channel with more permeable strata below elevation 540.

(4) Groundwater conditions. Groundwater levels at elevations 581 in 1969 and 576 in 1971 near Sta 133 (see (2) above) indicates a perched water table in the clay at a depth of 29 and 34 ft, respectively. Groundwater encountered at elevations 545 in 1971 and 550 in 1980 in the beach sand near Sta 133 was seepage water under pressure. An artesian condition exists in this sand because piezometers at this location (No. 37) and west of the site (No. 39) in the beach sand, showed a significant rise in water levels for several days after initial readings. This is confirmed by the local groundwater conditions (para 46) and by bailing tests in piezometers MCG-1, Nos. 40, 29, and 38 (Figure 21). The estimated groundwater profile before dewatering is a measure of the reduced artesian pressure in the beach sand caused by the distressed tunnel segments acting as a drain. This condition is evident in Figure 21 from the difference in profiles before dewatering, and 1300 to 1400 ft from the tunnel, the level in piezometers No. 37 near 15 Mile Road (Sta 137) and No. 43, 1284 ft east of Sta 101+40. The latter groundwater elevations correspond to the natural artesian pressures in the sand

away from distressed segments of the tunnel. This higher water profile represents conditions that could have existed prior to start of distress and tunnel cracking. The profile before distress could have been higher or lower at times from 1972 to 1980 depending on rainfall and recharge conditions as described in para 57. From the above conditions, it appears that the sand along the tunnel base and tunnel construction joints was subject to water pressure heads of at least 13 ft to 22 ft above the invert during the interim period from 1972 to 1980.

62. Tunnel and surrounding soil condition. Seepage and sand infiltration (piping), soil conditions behind lagging, joint conditions, and visual observations relating to geotechnical considerations are described in this section.

a. Sand piping and tunnel leakage. During the field visit of WES structures and concrete engineers (just after cleaning out of the tunnel south of break 1) on 15 May 1980, seepage water and fine sand were observed flowing in at the invert of a construction joint at Sta 120+71. The water pressure at this location was equal to a head of about 2.5 ft above the base of the tunnel (see Figure 21). A photograph of this condition is shown in Figure 22. This location is in the nondistressed segment where waterstops were not placed in construction joints and where sand exists along the invert as shown in Figure 23. A sample of the sand was obtained and the gradation is shown in Figure 24. This gradation is very similar to that for the natural sands along the base of the tunnel (see soil test results, para 64). When a concrete core at Sta 117+70 (23 May 1980), 2 ft above the invert on the east side, was taken (with the last 0.05 ft of concrete drilled using a 2-in. air hammer), water seepage began piping sand into the hole. The water pressure at this location was equal to a head of about 2.5 ft above the wood lagging at the base of the core. Again at Sta 117+25, water seepage piped sand into the tunnel after a concrete core at the invert was removed from the hole. On several occasions, a packer in one of the core holes near the invert was knocked out during bracing operations and water seepage piped 1 to 2 cu ft of sand into the tunnel. Seepage also occurred from 2-in.-diam holes drilled through the tunnel wall to check soil conditions. After completion, a wooden plug was inserted. Several days later seepage, as shown in Figure 25, was evident around the plugs south of Sta 122+82. When some of the

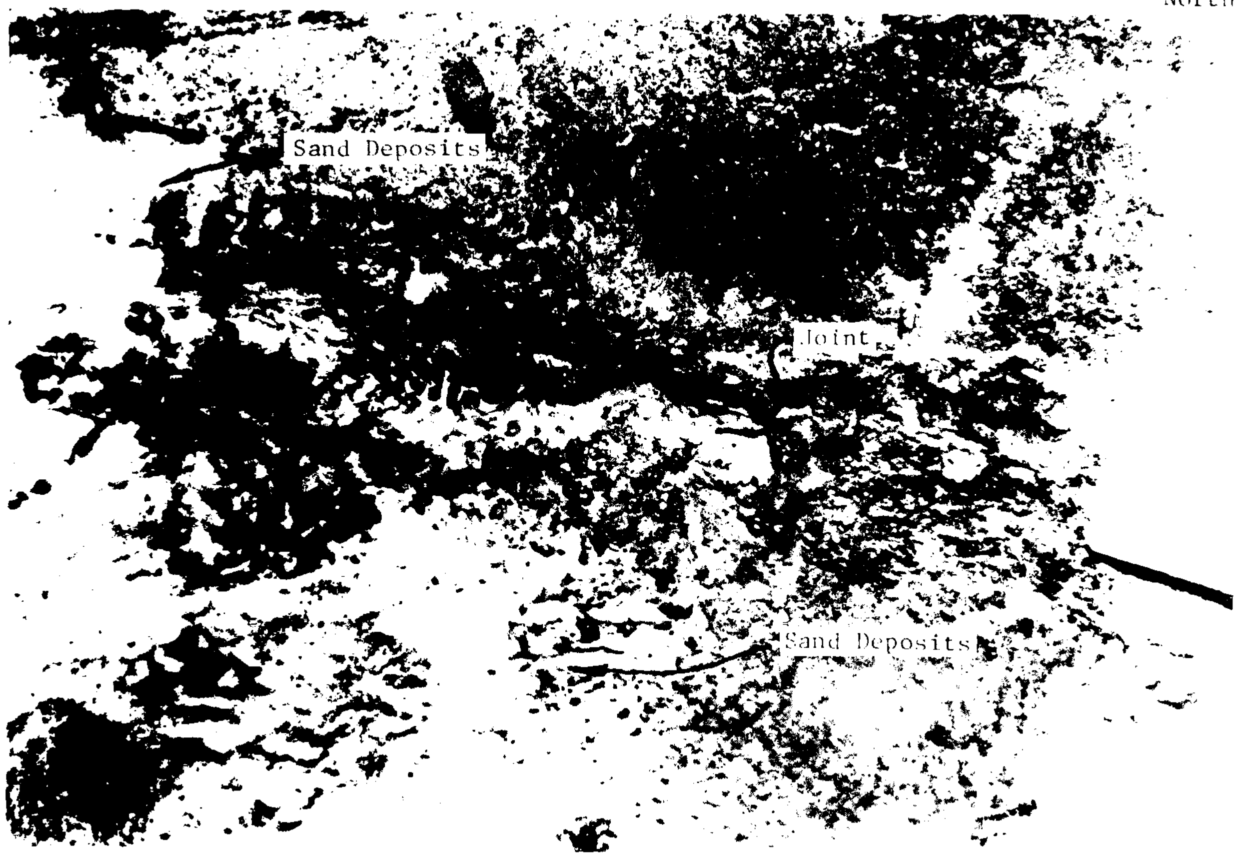


Figure ... Seepage water and fine sand flowing into tunnel at invert of construction joint, Sta 120+71, ... Tunnel Failure Study

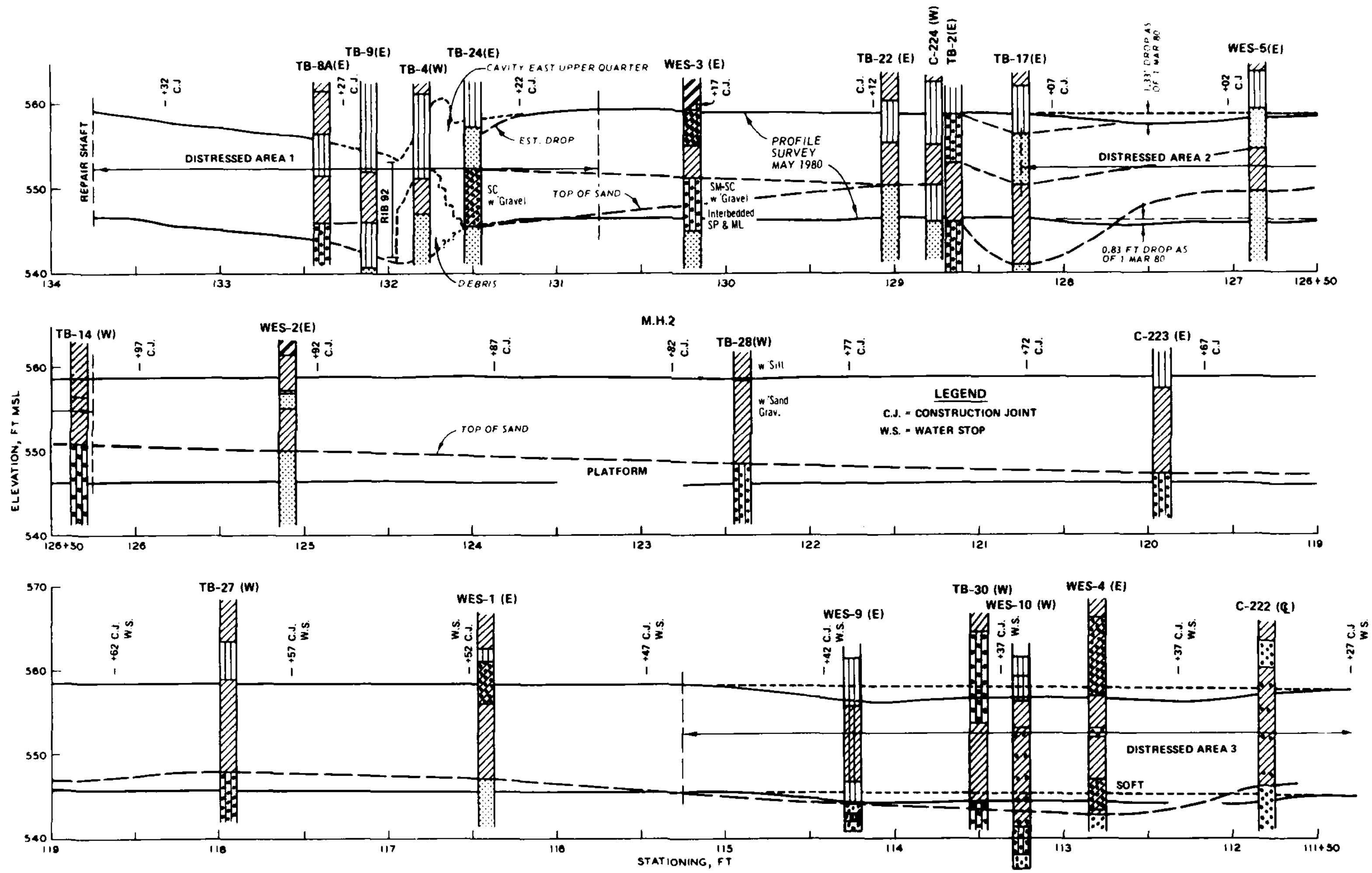
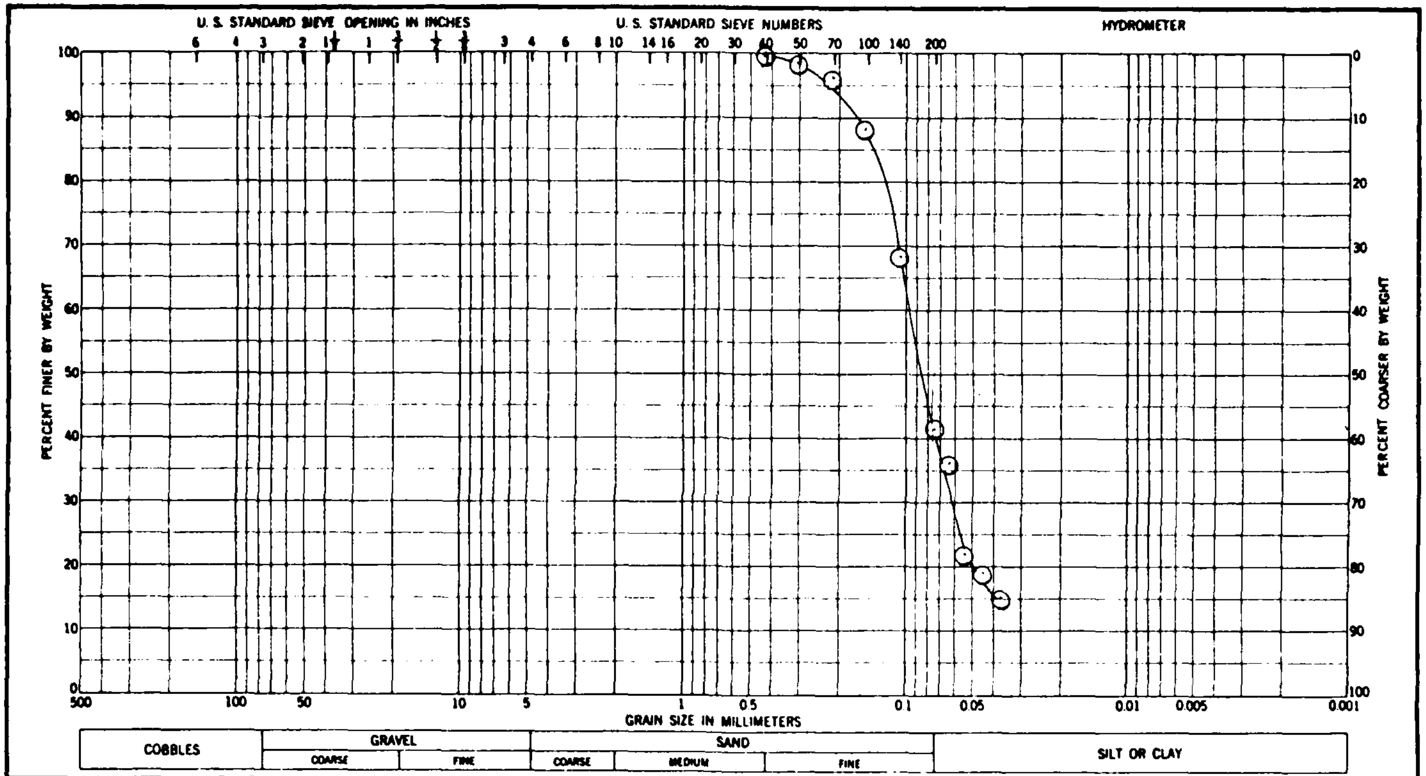


Figure 23. Summary of distressed tunnel profile and soil stratification, 15 Mile Road/Edison Corridor Tunnel Failure Study



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
		SILTY SAND (SM) GRAY				

GRADATION CURVES

Project DETROIT SEWER TUNNEL

Area SOLID MATERIAL FROM PIPING

Boring No. STA. 120 + 71

Date 10 JUL 80

ENG FORM 1 MAY 63 2087

Figure 24. Gradation of sand piped into tunnel at Sta 120+71, 15 Mile Road/Edison Corridor Tunnel Failure Study

Figure 25. Seeping water from plugged holes south of
Sta 122+22, 15 Mile Road/Edison Corridor
Tunnel Failure Study

plugs were removed, it was found that sand had been carried up into the hole by seepage water. The above incidents illustrate the piping characteristics of the beach sand under relatively low water pressure. Staining of the tunnel wall from water seeping out around the plugs and leaving an iron mineral deposit started several days after the holes were first plugged. Numerous construction joints and cold joints showed evidence of water leakage from staining and mineral deposits built up on the inside of the tunnel wall. An example of staining at a cold joint is shown in Figure 26. This area had waterstops in construction joints starting at Sta 118+62. Data summarized in Table 10 for the nondistressed area is indicative of conditions prior to distress in other areas. Water seepage was noted on 24 June 1980 at Sta 114+40 and 115+30 where the water pressure head was near the springline outside the tunnel. The water was clear with no apparent sand piping. The seepage at 114+40 was 1/2-in. high and coming through a narrow crack in the construction joint at the invert. At 115+30 water was seeping through a 1/8-in. crack in the west lower quarter.

b. Soil conditions behind lagging. The 2-in.-diam air hammer holes in the tunnel wall were made at selected locations in distressed and nondistressed areas initially to check for voids beneath the tunnel just below the location of clay-sand strata contacts derived from boring logs. However, no voids were found behind the lagging. Use of the small core penetrometer was then decided on after a number of the holes were completed. The cone penetration tests were made to obtain a qualitative indication of the consistency of clays (stiff to soft), and sands (loose to dense). The results shown in Table 11 verified soil types determined from surface borings. However, the results of these borings and tests are not entirely representative of natural conditions because of dewatering which increased the soil resistance, especially for sands. In the accessible area north of break 1, the sand, silty sand, and silty clay was frozen to semi-frozen from Sta 132+36 to 132+12 and the relative consistency was not considered. South of break area 1 no soft or loose conditions were found until Sta 128+39 near area 2. At several locations adjacent to construction joints in Distressed Area 2, 1.5 in. to 6 in. of penetration was recorded, indicating relatively loose sands and soft clays. Similar penetrations indicated loose silty sand and sandy clay at or near construction joints in the nondistressed segment at Stas 124+92,

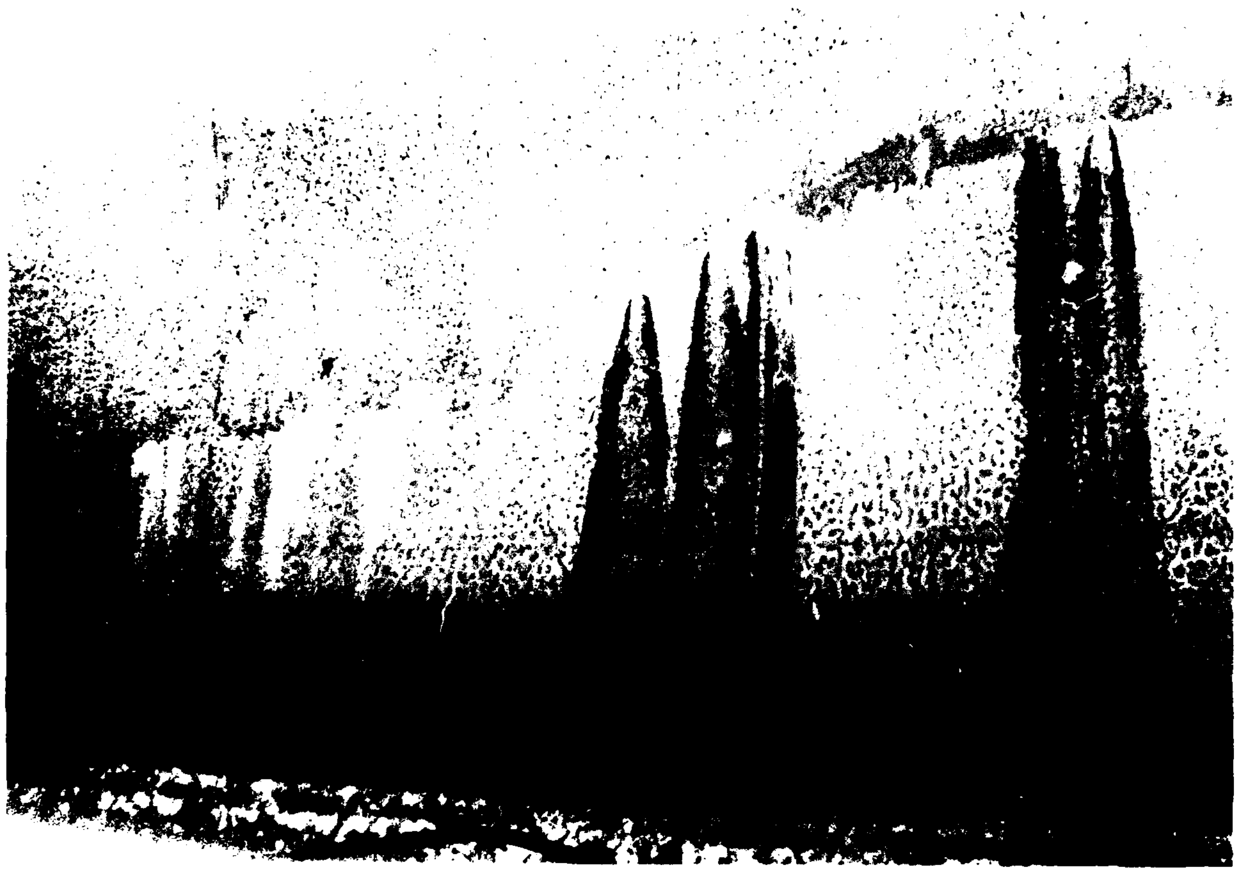


Figure 26. Example of staining indicating water leakage through cold joint, nondistressed tunnel section

123+87, and 122+82. Behind the lower quarter on the east side at Sta 123+07, very soft sandy silt was penetrated 18 in. At 122+82 very soft sandy clay was found just behind the lagging. In Distressed Area 3, two holes in the east and west lower quarter indicated silty sand to sandy silt. These soil types correlated with those found in boring WES-10. However, no apparent loose condition was noted. Additional holes in this area were not made because of the higher groundwater pressures and the possibility of excess piping of sand into the tunnel.

c. Joint conditions. Construction joints in the tunnel showed the following conditions:

(1) Pieces of construction joint from fractured areas in Distressed Area 2, Sta 128+07, east lower quarter and springline contained a firm deposit of silty sand on the joint face.

(2) A concrete core was taken at Sta 115±45 in the east side of the Sta 115+47 construction joint. The tunnel at this joint is shown in Figure 27. The core was taken to check the competence of an undistressed waterstop joint just north of Distressed Area 3. As shown in Figure 28, a waterstop was found in the joint keyway. Also found was a piece of wood at the lagging side of the joint. The wood did not appear to be part of the lagging and could be a piece of wood used to block off the bulkhead from the lagging during concreting. The joint was iron stained on the tunnel side portion, indicating water leakage. The joint was clean with no signs of silt and no signs of waterproofing mastic.

(3) At Sta 114+42, a small section of construction joint on the east lower quarter was filled with sandy silt behind a loose concrete cover piece about 4 in. thick. The area is shown in Figure 29. Subsequent digging into the silt indicated that it extended all the way through the joint. A waterstop and a piece of soft wet (partially decayed) wood was found in the digging process. Whether this wood was used to hold the waterstop out during concreting or was used to block off the lagging side of the concrete-form bulkhead is unknown. The waterstop and a piece of wood were also found high up in this joint.

(4) Construction joints at Stas 113+37, 112+32, and 111+27 in Distressed Area 3 were checked using a geologist's hammer and drill to open the joints. Waterstops and wood were also found as noted in Table 12.

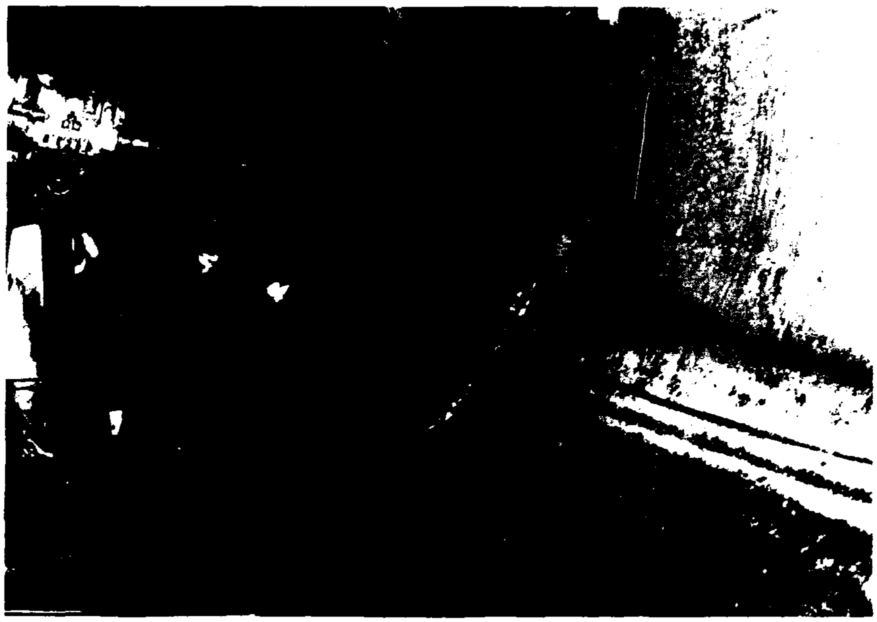


Figure 27. Location of concrete core at construction joint with waterstop, west side, nondistressed area, Sta [illegible]+[illegible], 15 Mile Road/Edison Corridor Tunnel Failure Study

Hole No.

DRILLING LOG	DIVISION Detroit	INSTALLATION 15 mile + Edison Conn'd	SHEET 1 OF 1 SHEETS

1. PROJECT Detroit Sewer	10. SIZE AND TYPE OF BIT 6 x 6 1/2
2. LOCATION (Coordinates or Station) Station 115+45 - West Wall Water Stop	11. DATUM FOR ELEVATION SHOWN (TBM or MSL) Bit No. 80PK1621
3. DRILLING AGENCY CSWSS	12. MANUFACTURER'S DESIGNATION OF DRILL Longyear
4. HOLE NO. (As shown on drawing title and file number) 115+45 - West No. 11	13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN — DISTURBED — UNDISTURBED
5. NAME OF DRILLER B Harrield	14. TOTAL NUMBER CORE BOXES
6. DIRECTION OF HOLE ☐ VERTICAL ☐ INCLINED ____ DEG. FROM VERT.	15. ELEVATION GROUND WATER
	16. DATE HOLE — STARTED 6/30/80 — COMPLETED 6/30/80
7. THICKNESS OF OVERBURDEN	17. ELEVATION TOP OF HOLE
8. DEPTH DRILLED INTO ROCK	18. TOTAL CORE RECOVERY FOR BORING %
9. TOTAL DEPTH OF HOLE	19. SIGNATURE OF INSPECTOR Gary J B Dub—

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
Waterstop	0.0		Break			NL Run 1.5'
			Fe Staining			Began 3:00 Rec 1.5'
			waterstop (Plastic/wick)			End 4:50 Loss —
						Time 1.5hrs Gain —
	1.0		Break			Drl time 1.5hrs
			Cleared			Hyd press —
			Run (0.0 to 1.5')			Water press —
			Wood			RPM 50 t
						Drl Action Smooth
	2.0					Water ret
						W. Brown
						Remarks

Figure 28. Log of concrete core of waterstop joint in nondistressed area

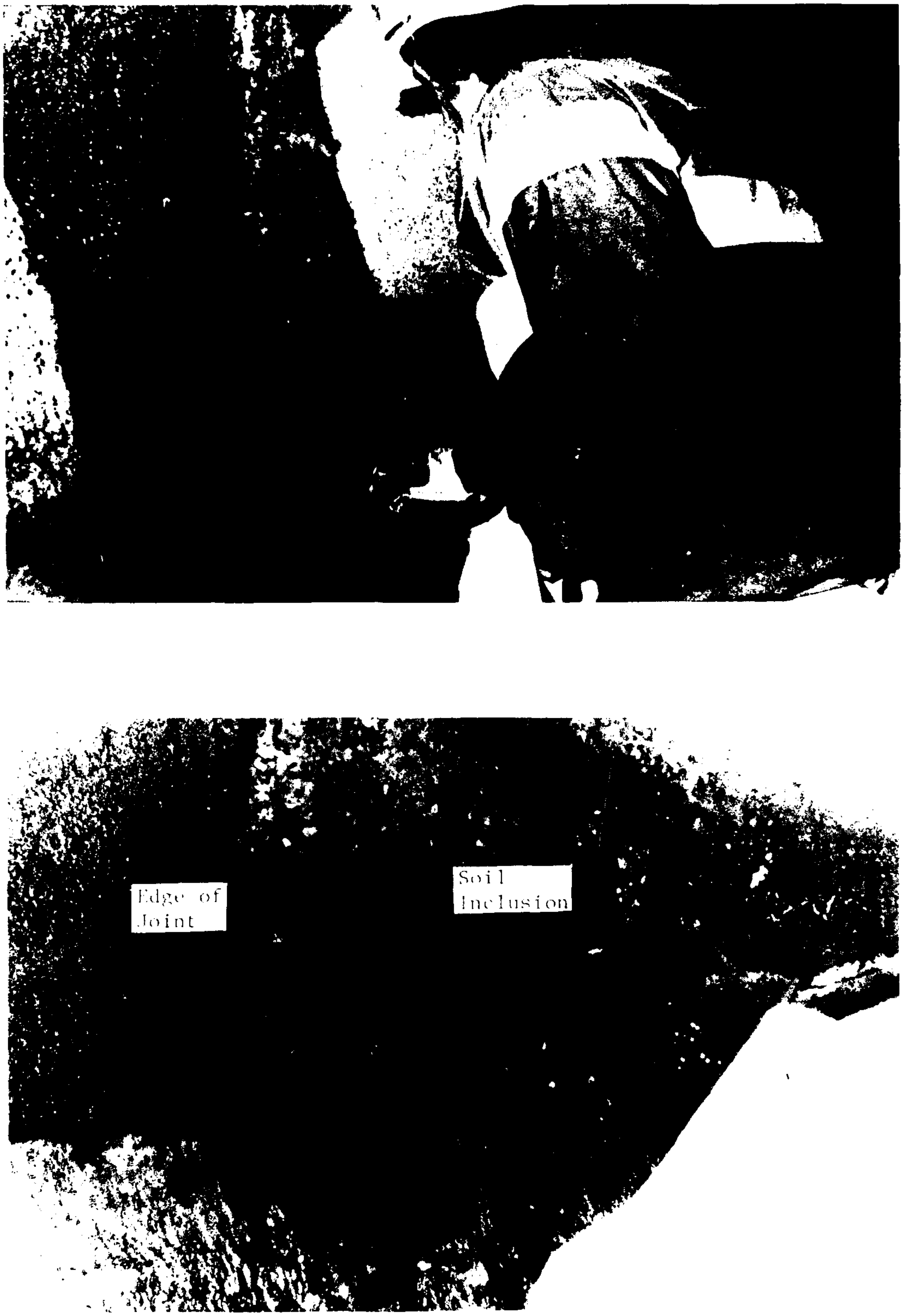


Figure [illegible]. Construction joint, east side Sta [illegible]

AD-A096 440 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY, DETROI--ETC(U)
JAN 81 D ALBERT, G C HOFF, B LORENCE NCE-IA-80-055

UNCLASSIFIED WES/TR/GL-81-2 NL

2 OF 5
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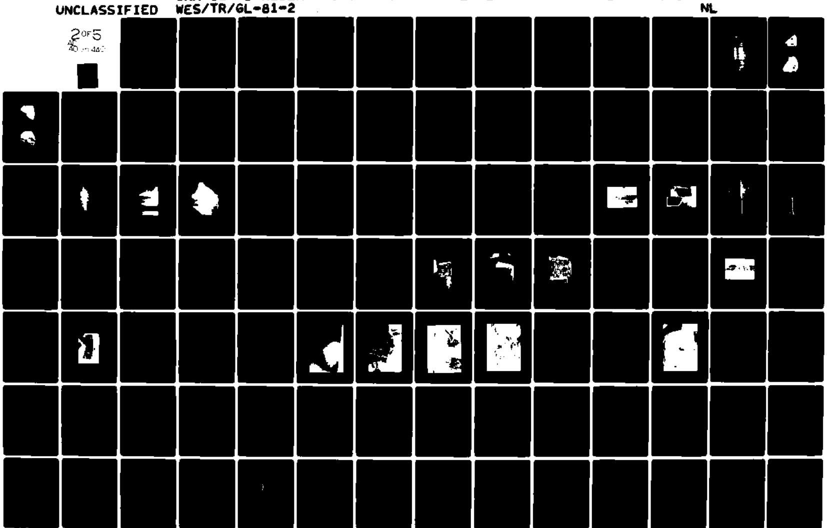


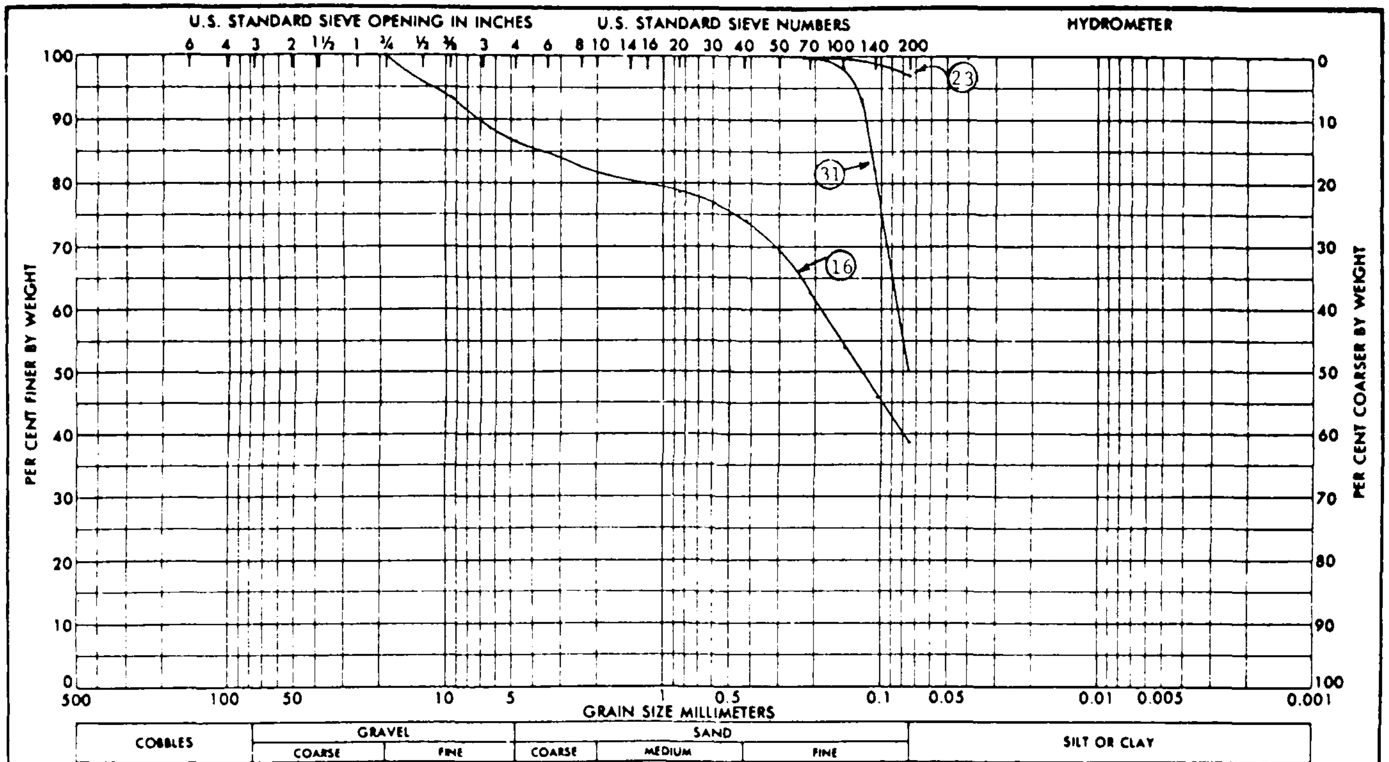
(5) A concrete core of a cold joint in the nondistressed area between Distressed Areas 2 and 3 was checked when received at WES. The core separated easily at the cold joint and contained a thin veneer of fine sand.

(6) The above information indicates that construction joints in Distressed Areas 2 and 3 contained silty sand, that very fine sand existed in cold joints in the nondistressed area, that the waterstop in a non-distressed area contained wood and experienced leakage. In addition, when the tunnel section south of Distressed Area 1 was cleaned out in early May 1980, a considerable amount of fine sand was removed. The competence of waterstops in construction joints in Distressed Area 3 could have been detrimentally affected by wood found in the keyways.

Soil properties

63. Soil properties from WES laboratory tests are summarized in this section. A test data summary sheet is shown in Table 13 and detailed test results are contained in Appendices B through E.

64. Soil classification and sand gradation. Laboratory classifications were performed on sections of undisturbed samples trimmed into specimens for different tests, and whole samples selected for physical property tests and special piping tests. The results indicated that soils at the site were more variable than indicated on graphical logs of borings. Gradation curves (Appendix B) summarized in Figures 30 through 32 and Atterberg limits (Table 13) show that some samples recovered from the beach sand unit classified in the field (from material in the ends of undisturbed sample tubes) as fine sand often contained more silty and clayey strata. This condition was previously noted in paragraph 60 from gradation data shown in Figures 14 through 18. However, jar samples from split spoon borings indicated strata of very fine sand similar to the gradation of sample 38, boring WES-1 (Figure 30). The sample of silty sand piped into the tunnel at Sta 120+71 (Figure 24) indicates that very fine cohesionless sand exists along the tunnel. The gradation of the fine sands is further illustrated in Figure 33 by the grain size distribution curves shown from the CS-858 contract drawings for the



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
16	52.0 - 52.9	Gravelly Silty Sand (SC-SM)		20	14	6
23	61.5 - 62.5	Silty Clay (CL) Gray				
31	74.4 - 76.0	Sandy Silt (ML) Gray				

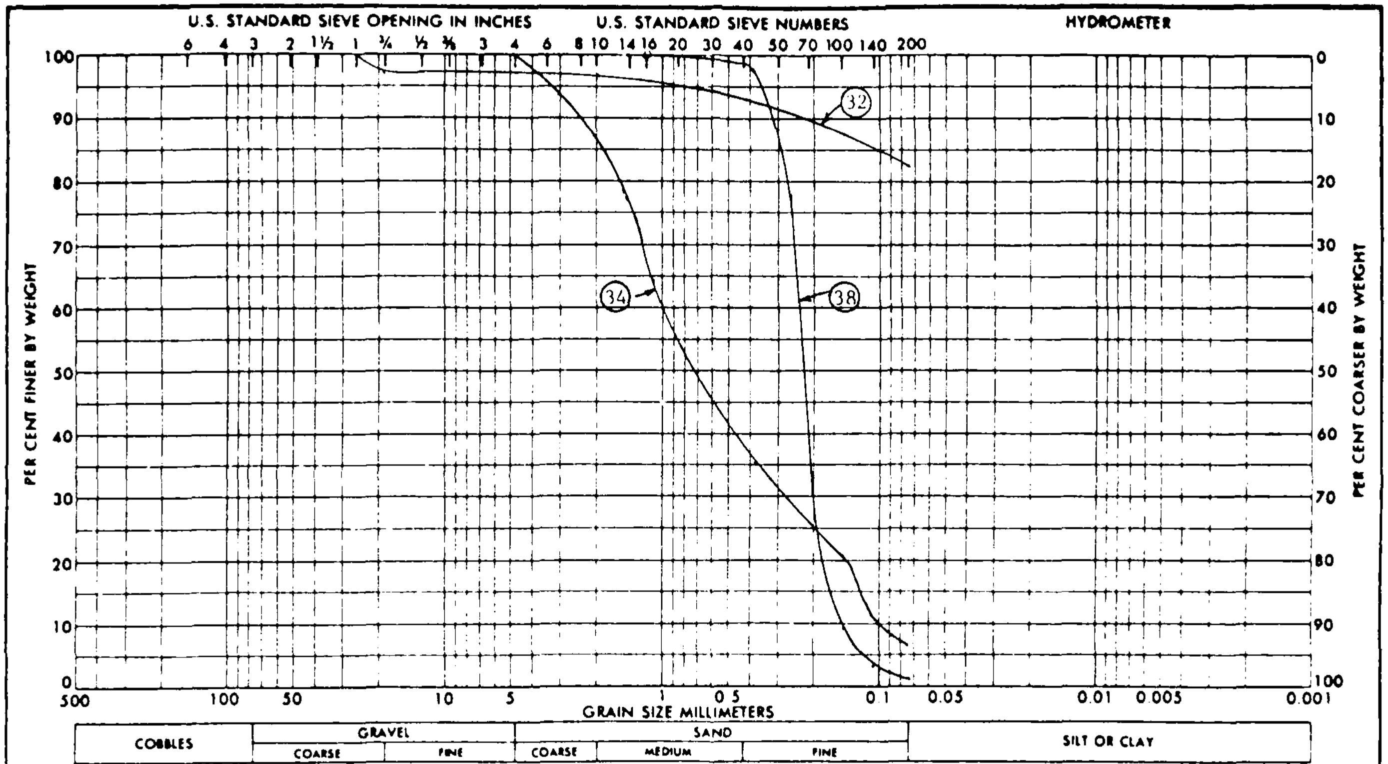
PROJECT	Detroit Sewer Tunnel
AREA	Sta 130+20
BORING NO	WES-3
DATE	11 Aug 80

GRADATION CURVES

ENG FORM 2087 1 MAY 63 REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE

U S GOVERNMENT PRINTING OFFICE 1963 OF—700-134

Figure 31. Gradation of samples from boring WES-3, 15 Mile Road/Edison Corridor Tunnel Failure Study



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
32	53.0 - 54.7	Sandy Clay (CL) Gray		27	15	12
34	59.5 - 61.1	Silty Sand (SP-SM)		NP		
38	63.9 - 65.9	Sand (SP) Gray		NP		

GRADATION CURVES

PROJECT Detroit Sewer Tunnel

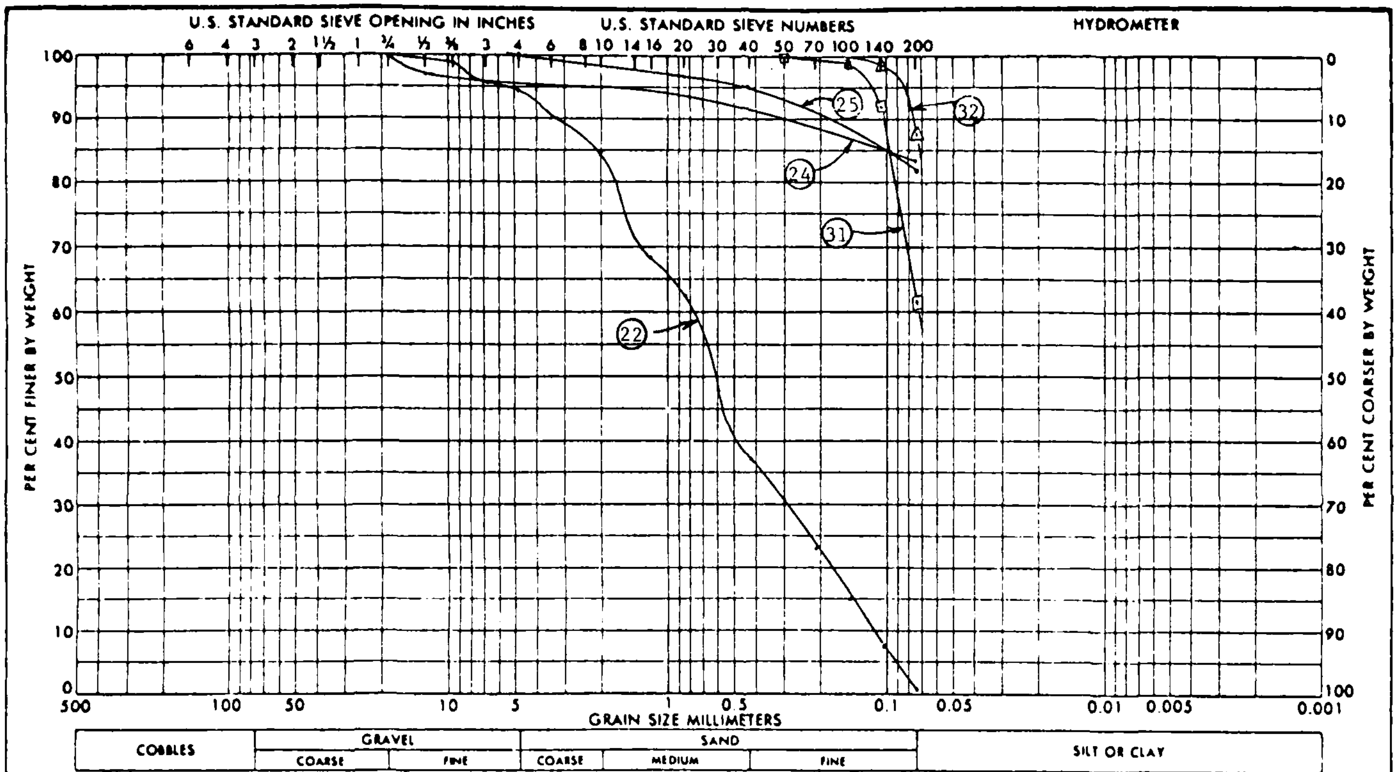
AREA Sta 116+40

BORING NO WES-1

DATE 11 Aug 80

ENG FORM 2087 1 MAY 63 REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE

Figure 30. Gradation of samples from boring WES-1, 15 Mile Road/Edison Corridor Tunnel Failure Study



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
22	47.5 - 48.2	Sand (SP) Gray w/Gravel		NP		
24	50.0 - 51.1	Sandy Clay (CL)w/tr Gravel		28	12	16
25	52.0 - 53.0	Sandy Clay (CL) Gray		24	12	12
31	66.5 - 66.8	Sandy Silt (ML) Gray				
32	69.0 - 70.0	Silt (ML)				

GRADATION CURVES

PROJECT	Detroit Sewer Tunnel
AREA	Sta 112+80
BORING NO	WES-4
DATE	11 Aug 80

ENG FORM 1 MAY 63 2087 REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE

U S GOVERNMENT PRINTING OFFICE

Figure 32. Gradation of samples from boring WES-4, 15 Mile Road/Edison Corridor Tunnel Failure Study

NEYER, TISEO & HINDO, LTD

GRAIN SIZE DISTRIBUTION CURVE

PROJECT NO. 77803 LAB SAMPLE NO. - SOURCE Test Boring Nos. 1, 5 and 21

PROJECT LOCATION 15 Mile and Edison Corridor FOR Tunnel Repair

BORING NO. 1, 5 and 21 FIELD SAMPLE NO. See Below SAMPLE DEPTH See Below SAMPLE ELEV. (TIP) -

SAMPLE DESCRIPTION Gray FINE SAND TO SILTY FINE SAND,

DATE SAMPLED Feb., 1980 BY NTH, Ltd. DATE TESTED Feb., 1980 BY NTH, Ltd. CHECKED BY -

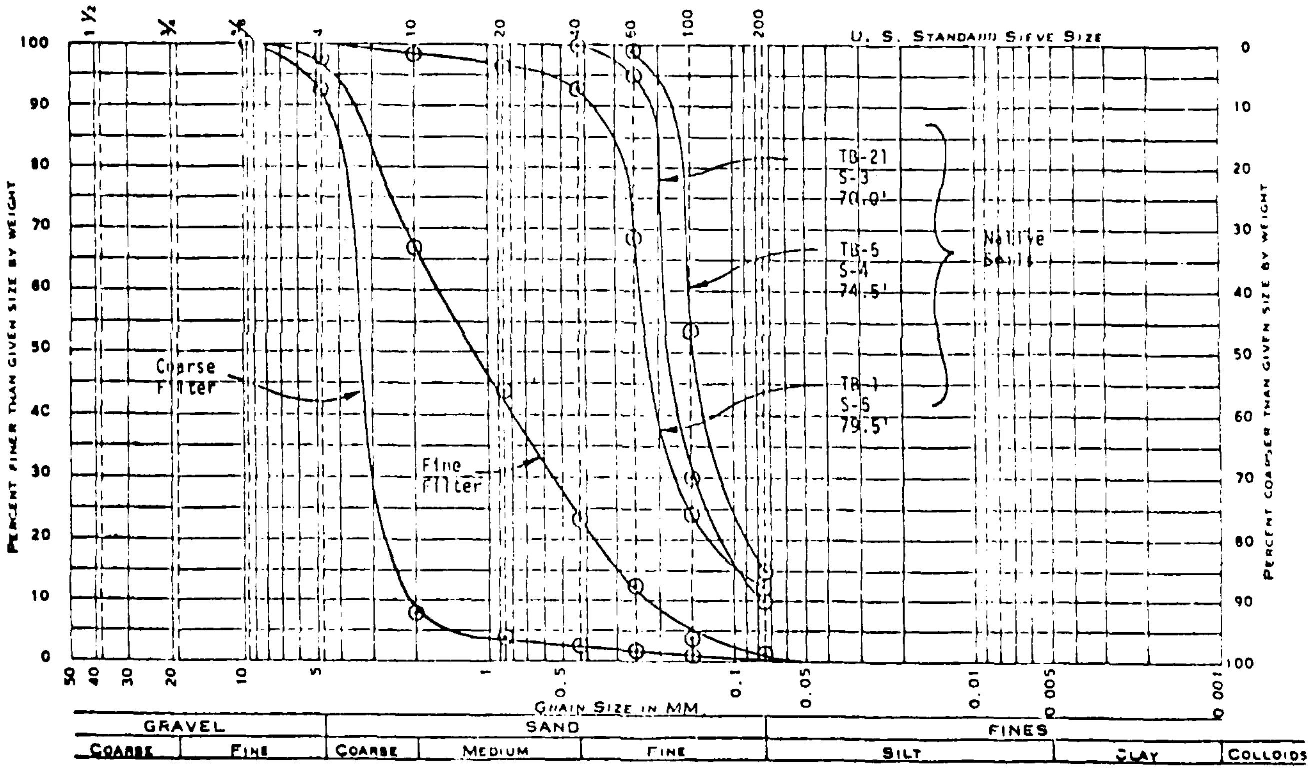


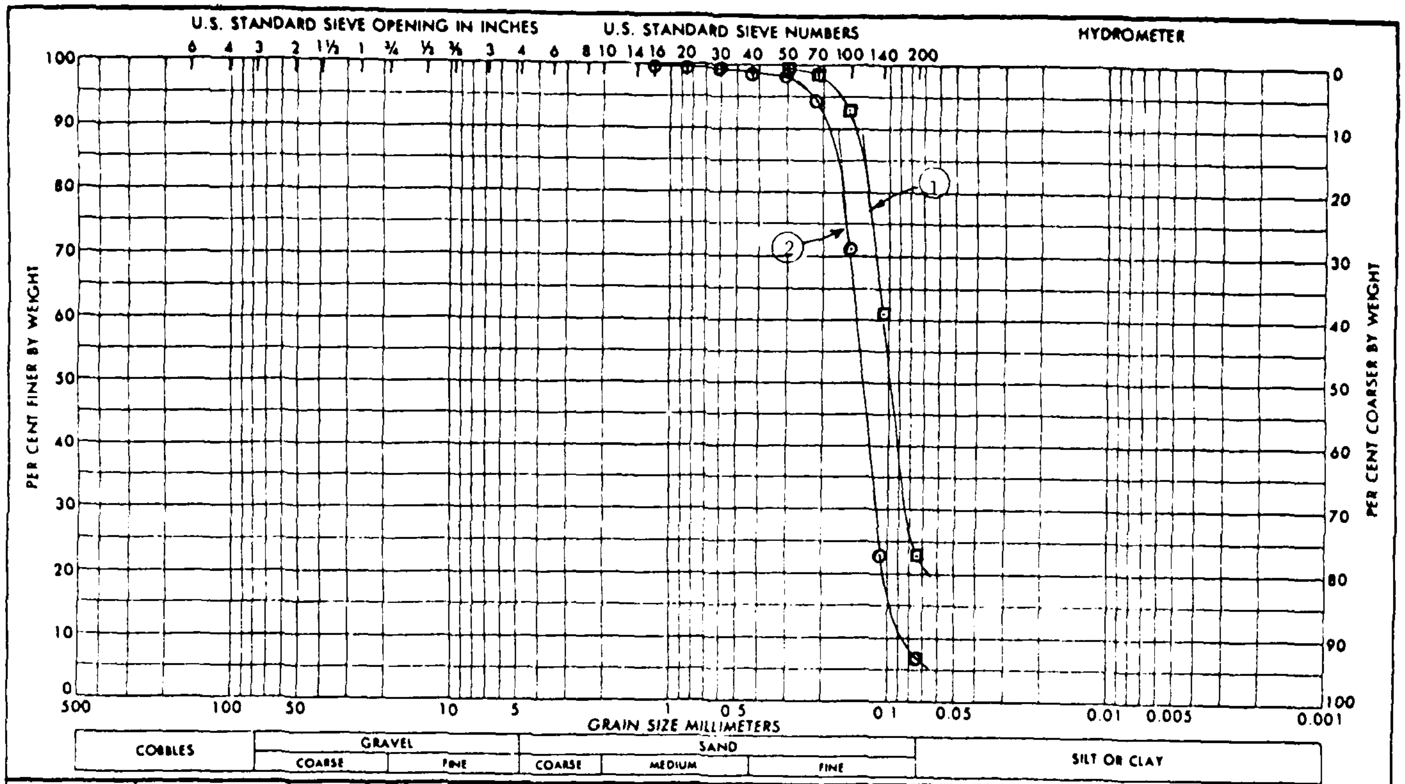
Figure 33. Gradation of samples from borings TB-1, TB-5, and TB-21, 15 Mile Road/Edison Corridor Tunnel Failure Study

temporary bypass and pumping shaft. The curves of interest are those labeled "native soils" for depths of 70 to 79.5 ft from TB-1, TB-5, and TB-21. These borings were located just north of and in break area 1. During clean out of break area 1, a large amount of sand was found under other soil and concrete in the tunnel. The lower segment of the tunnel at break 1 had settled about 5 ft into sand along the base of the tunnel. The gradations for two jar samples from the sand deposit in the tunnel are shown in Figure 34 and are similar to gradations for the silty sand from Sta 120+71, sample S-4 (TB-5) and sample 38 from boring WES-1.

65. Physical properties. Water contents, Atterberg limits, and wet unit weights for sele^^ed undisturbed samples from borings WES-3 (Sta 130+20) and WES-4 (Sta 112+81) are shown according to elevation in Figure 35. Specific gravities for the clay materials were uniform with values of 2.74 to 2.76.

a. Water content. The water contents of samples for Sta 132+20 were higher at depth than at Sta 112+80, perhaps due to the greater water seepage source from the dump material and the existence of more plastic clays at Sta 130+20. Water contents of samples from boring WES-4 (Sta 112+81) between elevations 590 and 555 were higher than those before construction at Sta 111+80 (boring C-222, Figure 10). These are the only two borings close enough to compare water contents after distress with those before construction. Thus, water contents after tunnel distress in area 3 appear to be higher than before construction. The water contents before tunnel distress could have been slightly higher since about two months of dewatering and drainage of soils around the tunnel occurred before boring WES-4 was made. In the upper 15 ft at Sta 112+81, water contents were near the liquid limits, indicating normally consolidated soils; at depths below elevation 590 water contents were close to the plastic limit, indicating an overconsolidated condition.

b. Wet unit weights. The total unit weights (as indicated by wet density) are shown in Figure 35 since they relate to the existing total overburden pressure. Wet densities of about 129 to 143 pcf for samples from Sta 130+20 (under the dump) are generally lower than



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
1	Upper Matl	Silty Sand (SM)				
2	Lower Matl	Sand (SP-SM)				

PROJECT 15 Mile Road/Edison Corridor Tunnel Failure Study

AREA Soil deposited in break area 1

BORING NO

DATE Sampled 21 July 1980

GRADATION CURVES

ENG FORM 1 MAY 63 2087 REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE

U.S. GOVERNMENT PRINTING OFFICE

Figure 34. Gradation of samples from soil deposit, break area 1, 15 Mile Road/Edison Corridor Tunnel Failure Study

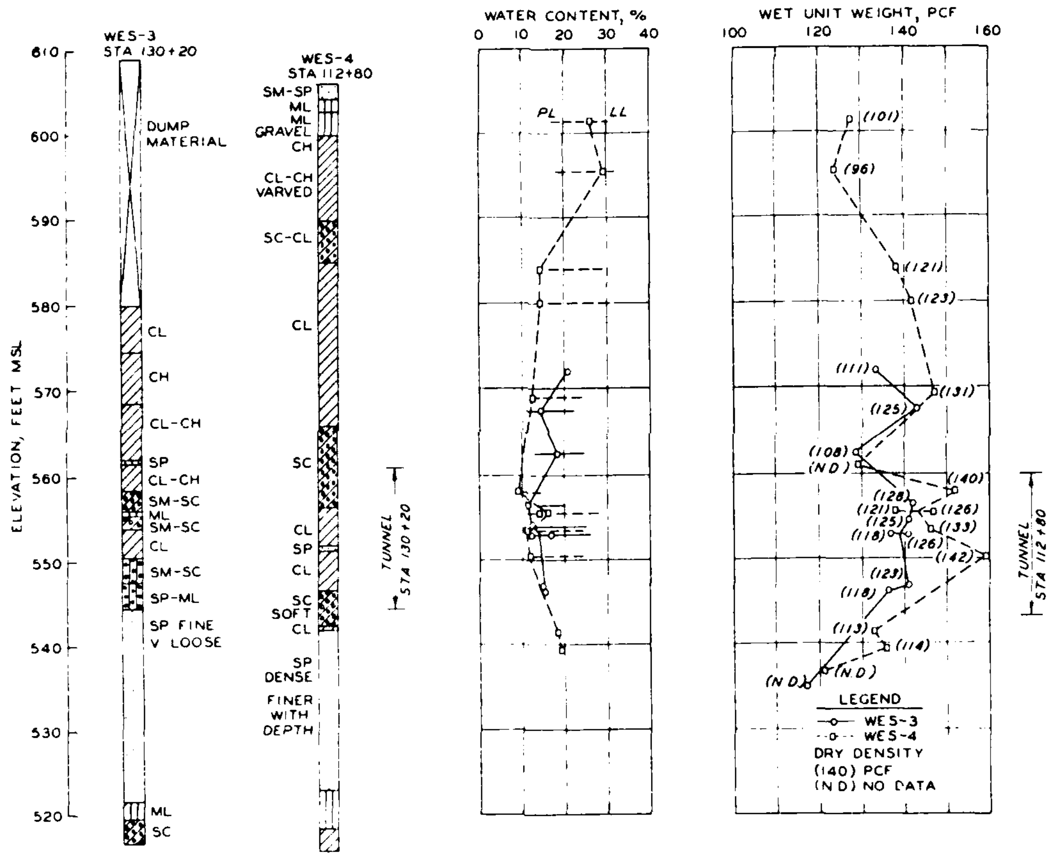


Figure 35. Water content, plasticity, and wet unit weight versus depth

those at Sta 112+81 (dry densities also are lower). The wet densities at Sta 112+81 increase with depth to 152 pcf (at the tunnel crown in Distressed Area 2), then show a marked decrease for the sandy clay above the springline. However, this decrease may be a result of natural variation since unit weights for samples from boring WES-4, between elevation 553 to 558 (Figure 35), are almost the same as those for samples from boring C-222 (Figure 10). The density of the silty sand samples (below elev 539, Figure 35) are not as dense as would be expected from the blow count data. Since sampling causes stress release, the samples tend to expand and may be further disturbed during handling and transit to the laboratory. This is especially true for silty sand and sand samples. Consequently, unit weights can be lower than they are in situ. When clay samples were opened in the laboratory, some were noted to have cracks indicative of stress release. Consequently, all test results should be considered with the above in mind.

66. Special piping tests. To assess the piping susceptibility of sands and silts around the tunnel, six undisturbed samples were tested. Five samples were selected from those recovered from the sand unit along the base of the tunnel and one from the sandy clay strata at the tunnel crown. The selection of samples was limited since the fine-grained sands along the tunnel and in the beach sand unit were easily eroded out of the tubes during field sampling. Recovery was low, in spite of attempts to obtain complete samples (using core catcher strips in 5-in.-diam tubes, heavier drilling mud, and pushing 3-in.-diam sample tubes). Although most samples tested had homogeneous pockets of sand, they also had varying amounts of silt and clay. The samples tested and a summary of test results are listed in Table 14.

a. Initial tests. The first two samples were tested with a 1/16-in. wide longitudinal slot cut along the central one-third of the sample. These samples piped material next to the sample tube interface where the least resistance to flow occurred. To provide better distribution of seepage piping, the longitudinal slots were sealed and five short circumferential slots, 1/32 in. wide, were cut around the middle of these samples and the samples retested. When a sample preserved in wax and cardboard tube (WES-4-20) was tested, the slots

tended to swell shut and further tests were restricted to samples in steel tubes.

b. Piping sequence. When the water pressure was raised to 2 or 3 psi at the start of a test, soil particles began streaming out of the lower slot as shown in Figure 36. More material usually flowed out of slots in the sides than out of the slot in the top. Very little material flowed out of slots in the bottom, probably due to the weight of the sample against the bottom of the tube. Under a given pressure, piping ceased after several minutes. If the water pressure was then shut off, allowing "soil readjustment" for several hours (at zero pressure), a small amount of piping could be reinitiated when the same pressure was again applied. Piping occurred readily from samples containing less dense sands and silts having little or no cohesion.

c. Sample erosion. Examination of samples after testing revealed erosion primarily along the top half of samples and also along the sample and tube interface. An example is shown in Figure 37. Samples tested were not tight against the tube. Erosion along an interface (such as between the tunnel lagging and adjacent soil) would be expected because of the likely existence of small voids. This condition would allow water under pressure to seek the path of least resistance, picking up soil particles along its route. An anomalous erosion path occurred in sample WES-4-32, as shown in Figure 38. This sample developed a circular piping hole that continued to the slots at the middle of the sample. During testing, a sudden rise in water pressure was made inadvertently. It is suspected that this pressure jump started the hole in the center of the sample. However, the other end of the sample eroded in the usual manner.

d. Piping susceptibility. Because of the heterogeneous quantities of sand, clay, and silt in the samples and variation in erosion, a quantitative scale of piping susceptibility for the sands and silts cannot be established. However, it was evident that the sands and silts readily piped under water pressures as low as 3/4 to 2 psi. This pressure is equal to a head of water of only 1.7 to 4.6 ft. The first sample tested was predominantly fine- to medium-grained sand and began piping rapidly under the low water pressure of 3/4 psi. The gradation of the piped material versus material left in the sample is shown for tests on sample WES-1-34 and WES-1-38 in Figure 39. The piped material gradation is similar to silty sand piped into the tunnel (Figure 24).

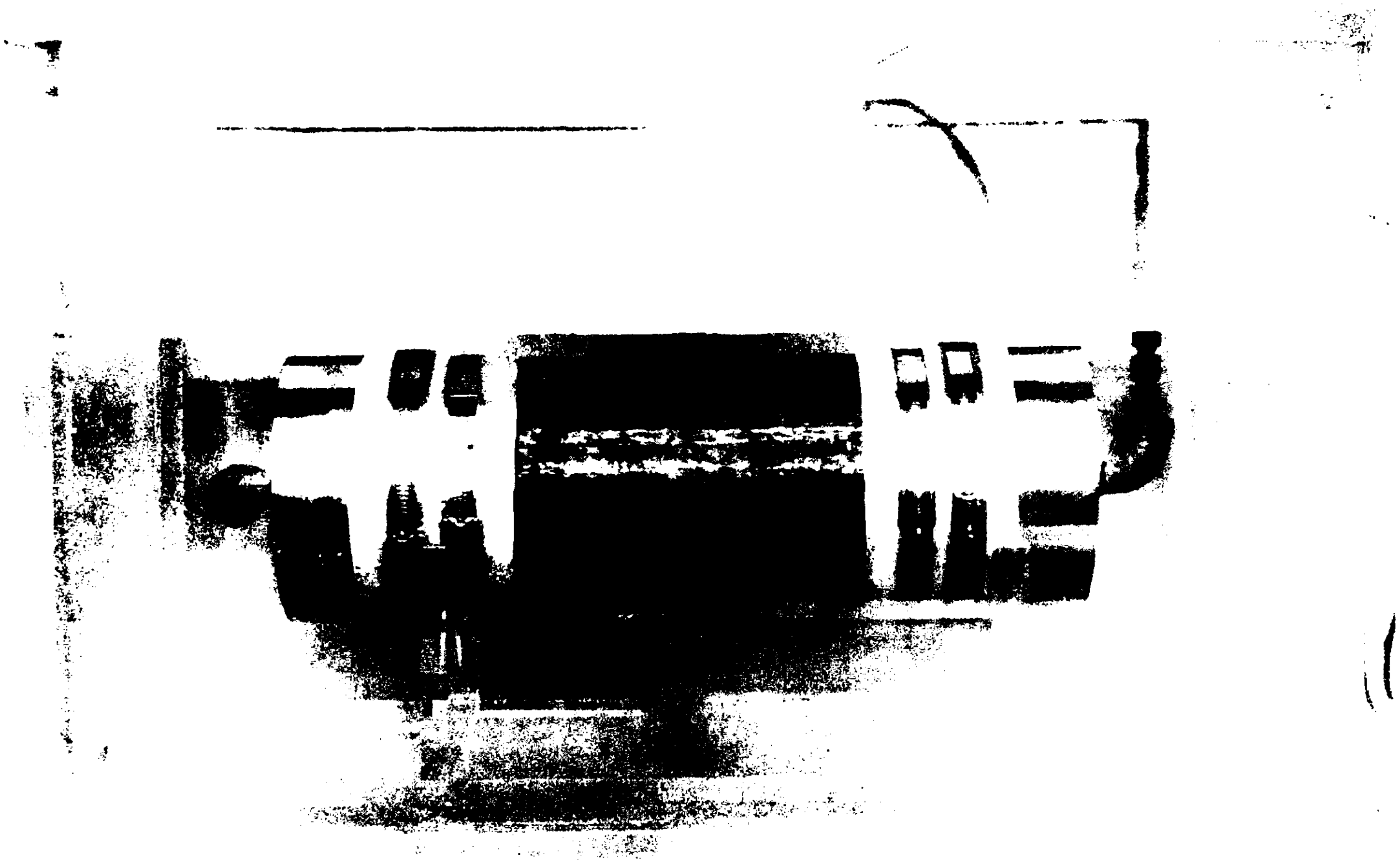


Figure 46. Special piping test in progress, 15 Mile Road/Edison Corridor Tunnel Failure Study

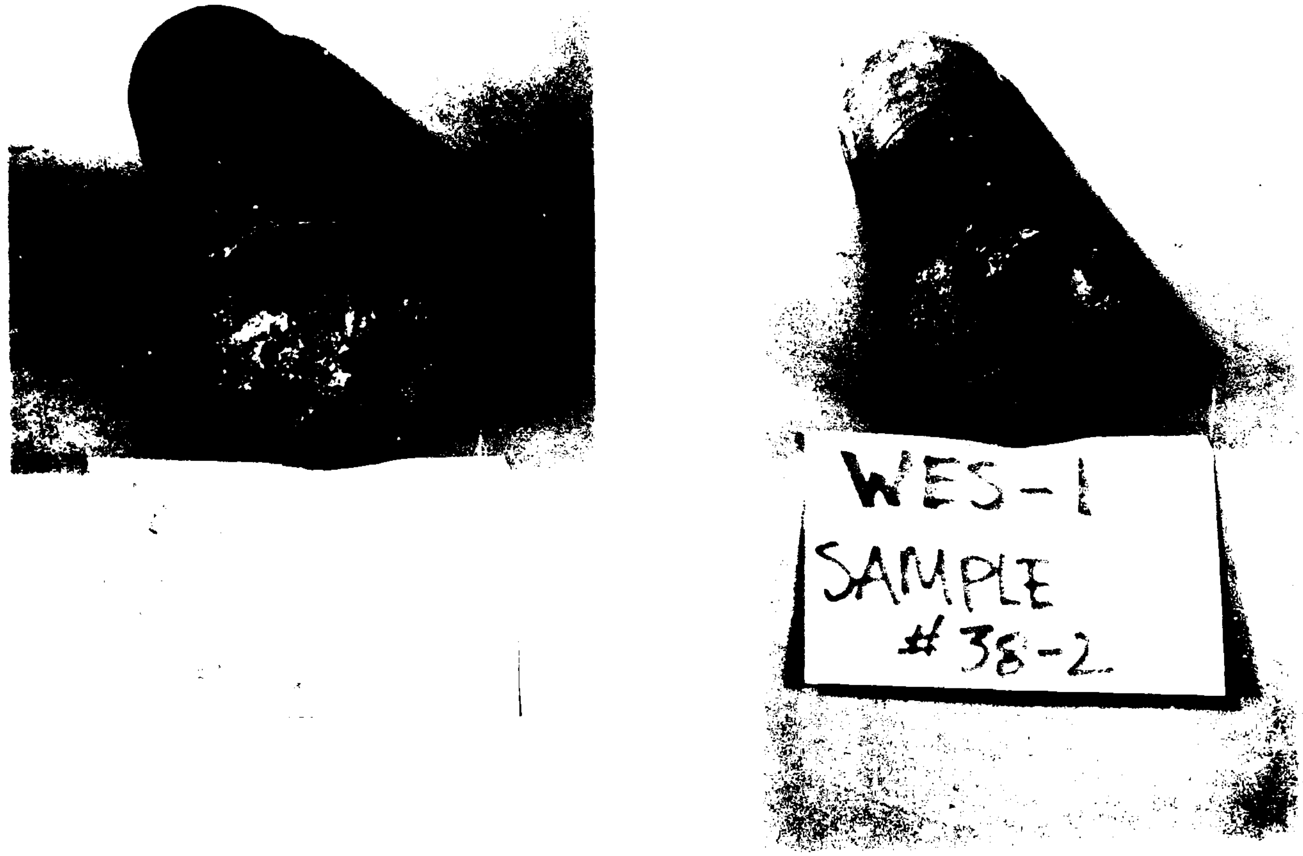
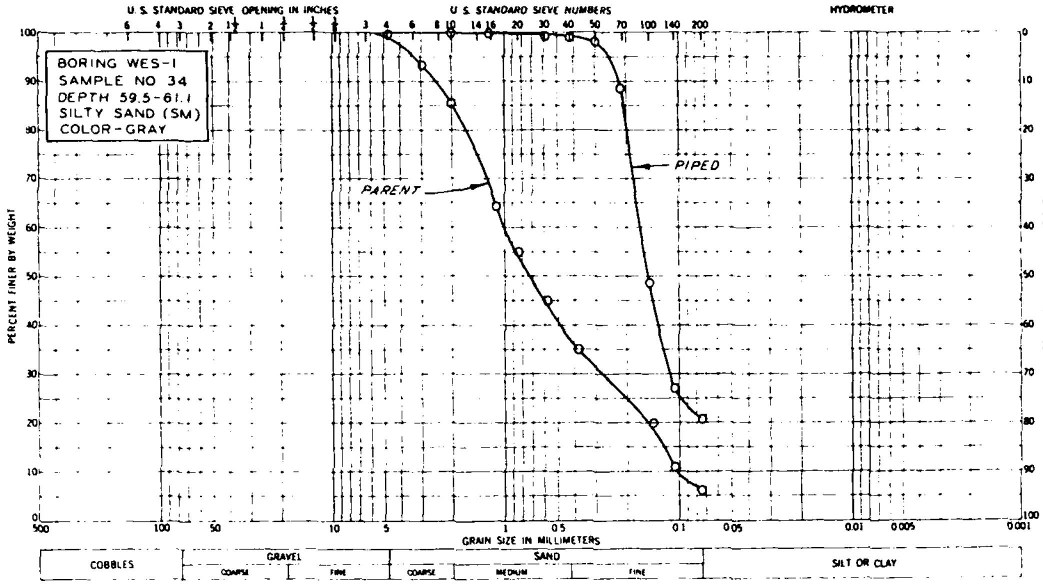


Figure 37. Erosion of sample after special piping test, 15 Mile Road/Edison Corridor Tunnel Failure

Figure [illegible]. Piping hole in sample after test, 15 Mile Road/Edison Corridor Tunnel Failure Study



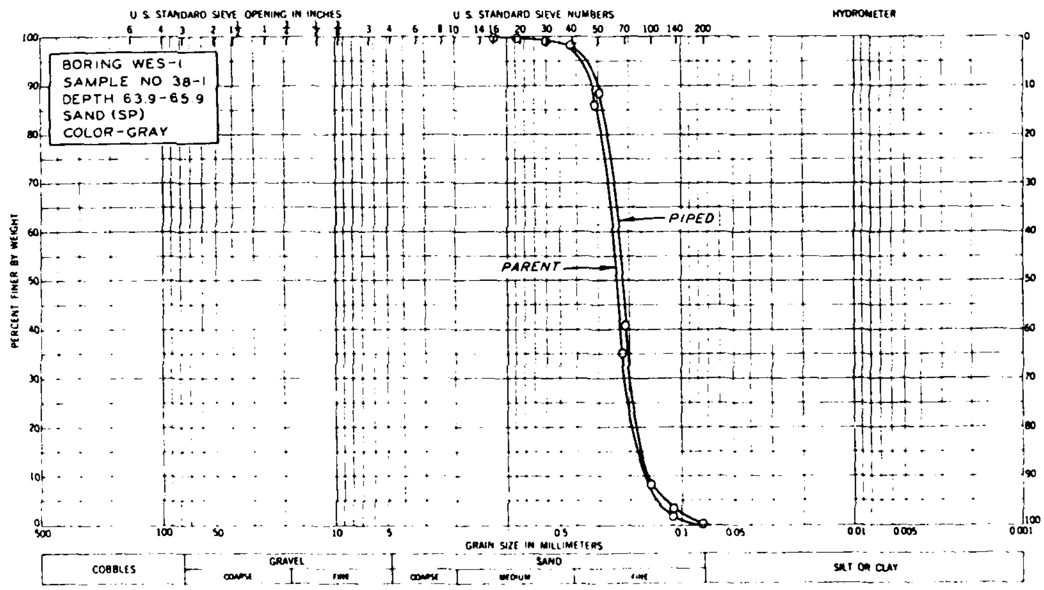


Figure 39. Comparison of parent and piped material gradation

Slot openings of 1/16-in. and 1/32-in. were selected as a practical minimum corresponding to possible construction joint openings. While these slot widths would not apply to the much smaller openings in cold joints, the minimum size of joint openings for the susceptible sands and silts with little or no cohesion can be estimated from criteria used for slotted drain pipes. To prevent soil infiltration, the slot width for drain pipes used to collect subsurface seepage is required to be equal to or less than the D_{50} size (mean particle size) of the soil next to the slot (Department of the Army TM 5-818-1). For the gradations shown in Figures 33 and 38, the D_{50} size varies from 0.15 to 0.6 mm and corresponds to a slot width of 0.006 to 0.02 in. Based on this criteria, fine sands and silts could be piped through joint openings as small as 0.01 in. which could correspond to cold joint openings.

e. Pinhole erosion tests. Results of pinhole tests on silty clay and clayey silt in a sample from the tunnel depth (WES-1-32, 53 to 54.7 ft) indicated no susceptibility to piping erosion. This result would be expected for cohesive soils.

67. Consolidation characteristics. Specimens trimmed from the two samples of clay (WES-4-12, 21 to 22 ft, and WES-3-18, 55.5 to 55.6 ft) used for consolidation tests (Appendix C) showed the clay to be overconsolidated. The estimated overconsolidation ratios (OCR) were about 3.8 for the clay at a depth of 21-22 ft and 1.4 for the clay at a depth of 55 ft. Based on a relationship of K_o versus plasticity index (PI = LL-PL) and OCR by Brooker and Ireland (1965), the estimated K_o values were 0.8 for the 21-22 ft depth and 0.43 for the 55-ft depth. The higher value at the 21-22 ft depth may be related to overconsolidation from dissication (para 40). Constrained modulus values were determined for the estimated in situ vertical effective stress by plotting the consolidation load and deformation as a stress strain curve. The estimated values were 2570 and 2890 psi, respectively for the depths of 22 and 55 ft. These values correspond to a drained condition with no lateral yield.

68. Drained direct shear strength. Standard direct shear tests (Appendix D) were performed on specimens from samples of lean clays and sandy clays from depths of 41 ft in boring WES-3 and 10, 25, 50, and 52 ft in WES-4. Each test included three specimens consolidated under a range of loads spanning the estimated in situ vertical pressure at the sample depth. The shear stress peaked and then decreased with increasing horizontal deformation, indicating a brittle type material in which the shear strength decreases after a small displacement. As listed in Table 13, the lowest strength ($c = 0.1$ tsf, $\phi = 24$ deg) is for clay at a depth of 10 ft. The strength parameters below the dump at a depth of 41 ft were $c = 0.15$ tsf and $\phi = 26.5$ deg. For the sandy clay at the 50 to 52 ft depth near the tunnel crown in Distrssed Area 3, strength values were $c \approx 0$, $\phi = 28.5$ deg and 33.5 deg. The strength of clay below the dump just south of Distressed Area 1 for the one test is slightly lower than that in Distressed Area 2.

69. Results from $K_o \bar{R}$ tests. The results of special $K_o \bar{R}$ triaxial compression tests (Appendix E) on four 2.8-in.-diam specimens are summarized in Table 15. After K_o consolidation to the estimated in situ vertical stress using stress controlled loading, the axial loading was increased with no drainage until a straight line on a Mohr stress plot was well established. Induced pore water pressures during undrained loading changed from positive to negative and the line of constant slope indicated a negative cohesion for three of the tests (WES-3-24, WES-4-24, and WES-4-30). For the three tests, the failure condition was taken at zero induced pore water pressure and zero cohesion to establish a ϕ value.

a. K_o values. The values of K_o appear reasonable since they tend to agree with the value of 0.43 based on the consolidation test on a specimen from sample WES-4-24 which was also used for one of the $K_o \bar{R}$ tests. Thus, it appears that the in situ K_o values after tunnel distress and dewatering are in the range of 0.4 to 0.45. A range of 0.4 to 0.5 is considered reasonable for conditions prior to distress.

b. Modulus values. The deviator-stress-strain curves for undrained loading (in which the effective and total deviator stress values are equal) showed an increase in slope at strains where induced pore water pressure became negative. The modulus values were determined from the initial segment of the stress-strain curve and show a lower value for the silty clay (CL) specimens than for the silt and interbedded sand-silt specimens. This appears reasonable since values for less cohesive soils such as dense silts and fine sands are usually higher. Data in Table 16 for similar glacial lake materials, show a range of modulus values estimated for this report from the results of two standard $\bar{R}$ triaxial tests reported in the Midland FSAR. Only materials below a depth of about 180 ft (elevation 420) at the Midland site were reported to have been heavily overconsolidated by ice loading in past geologic history. Ground surface at Midland is at about elevation 600 and corresponds with the tunnel site ground surface elevations and confirms that overconsolidation of materials above elevation 520 at the tunnel site is not due to ice loading. It can be noted from Table 16 that while cohesion values were obtained in these tests, the $\bar{\phi}$ values are similar to those for the $K_o \bar{R}$ tests in Table 15.

Analytical studies

70. Analyses were made to estimate in situ vertical and lateral pressures for use in the structural finite element analyses in the subsequent structural section of this report. The range of values of effective and total stress earth pressure coefficients, Poisson's ratio, and modulus of elasticity applicable for soils at the site were also determined. Changes in the in situ stresses after restoration of the groundwater and potential water and soil infiltration into the tunnel were also analyzed. These analyses are summarized below.

71. In situ stresses. Vertical and horizontal pressures were estimated for two conditions on the basis of wet unit weights from undisturbed samples and K_o values from $K_o \bar{R}$ tests. The first condition, as shown in Figure 40 at boring WES-3, Sta 130+20 (just south of Distressed Area 1), is during mining with dewatering. The second is before and after construction when the water table returned to a

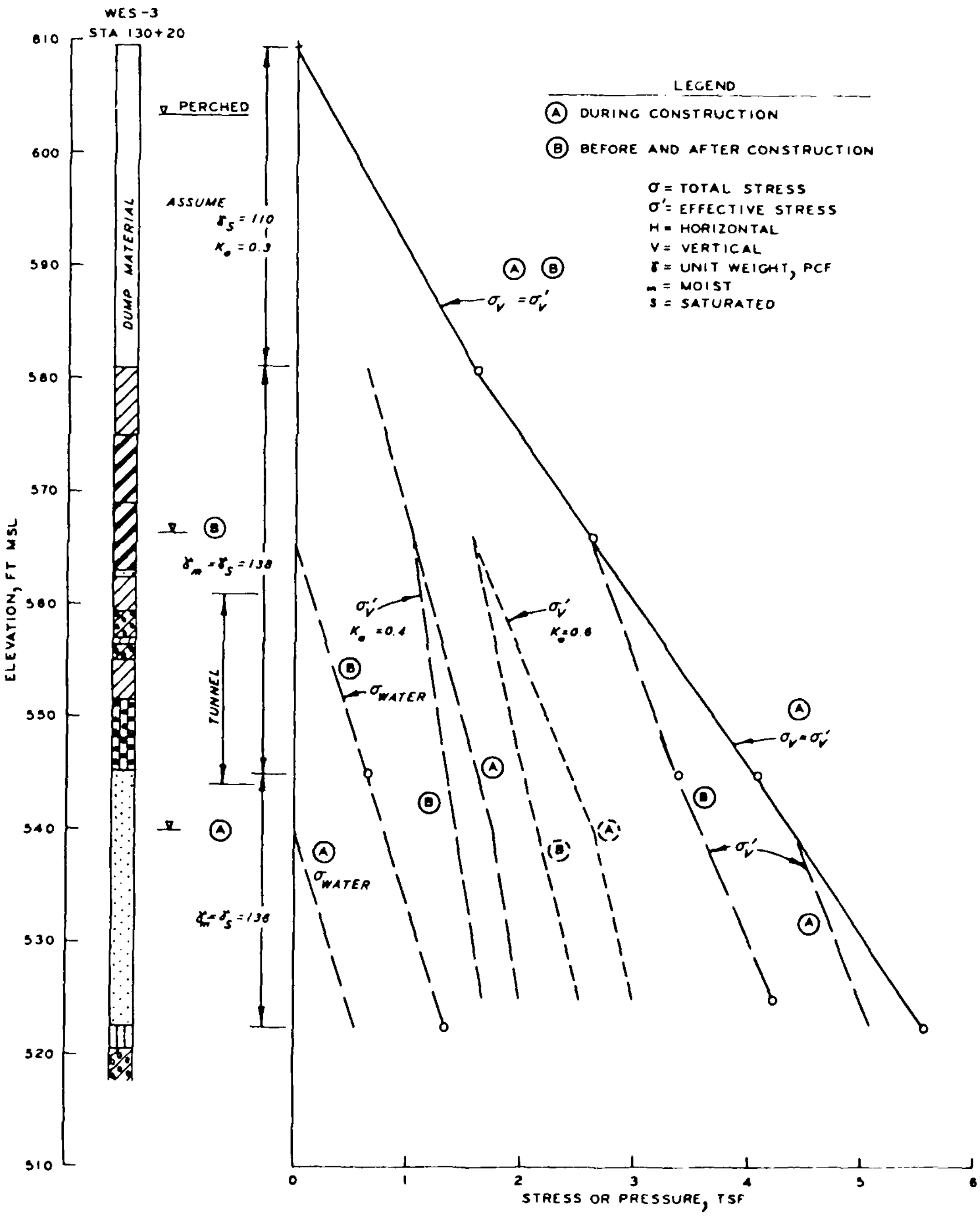


Figure 40. Estimated in situ soil stresses, Sta 130+20

natural condition at about elevation 566 (based on measured piezometric levels 1300 to 1400 ft from the tunnel (para 61 g).

a. Vertical pressures. Total vertical stresses are based on total unit weights. A perched water condition exists in the dump area and the water and material in the dump rest on the underlying soils as if contained in a bathtub. Drainage of the dump is prevented by the underlying plastic and impermeable clays. The perched water level is based on the log of boring TB-24 that showed fill with some debris to a 6-ft depth and that groundwater was encountered at the same depth (considered reliable because the hole was advanced with a solid stem auger to 10 ft and cased to 12.5 ft). The saturated unit weight of 110 pcf for dump material was based on the loose, caving conditions in boring WES-2 (para 60). Effective vertical pressures are based on submerged unit weights of soils below the water levels shown for conditions A and B. The artesian pressure acting in the beach sand unit is assumed for condition B to rise through the lean clay and sandy clay which is stratified with silt and sand seams. A similar plot of vertical pressure versus depth is shown in Figure 41 for Sta 112+81 (adjacent to Distressed Area 3). Vertical stresses are only about 0.3 tsf higher than at Sta 130+20 and the stresses shown for Sta 112+81 are representative of stresses at locations other than the dump area.

b. Horizontal stresses. Effective horizontal stresses are equal to K_o (the effective stress coefficient of earth pressure at rest) times the effective vertical stress. The effective stress is equal to the total stress less the pore water pressure. For the drained condition with no pore water pressure during dewatering, the effective stress was taken as equal to the total stress. The values of K_o shown in Figures 40 and 41 are based on the K_o $\bar{R}$ test results (Table 15) and K_o values estimated from consolidation test data. The values are affected by stress release of ground pressure during sampling and may be low. Consequently, effective horizontal stress lines for an upper limit for K_o of 0.6 are shown on Figures 40 and 41 to bracket possible values. A value of 0.65 was determined for very dense highly overconsolidated glacial till north of Toronto (Radhakrishna and Klym, 1974) and a value of 0.6 is considered an upper limit for the dense materials at the tunnel site. When total stresses are considered beneath the water table for saturated

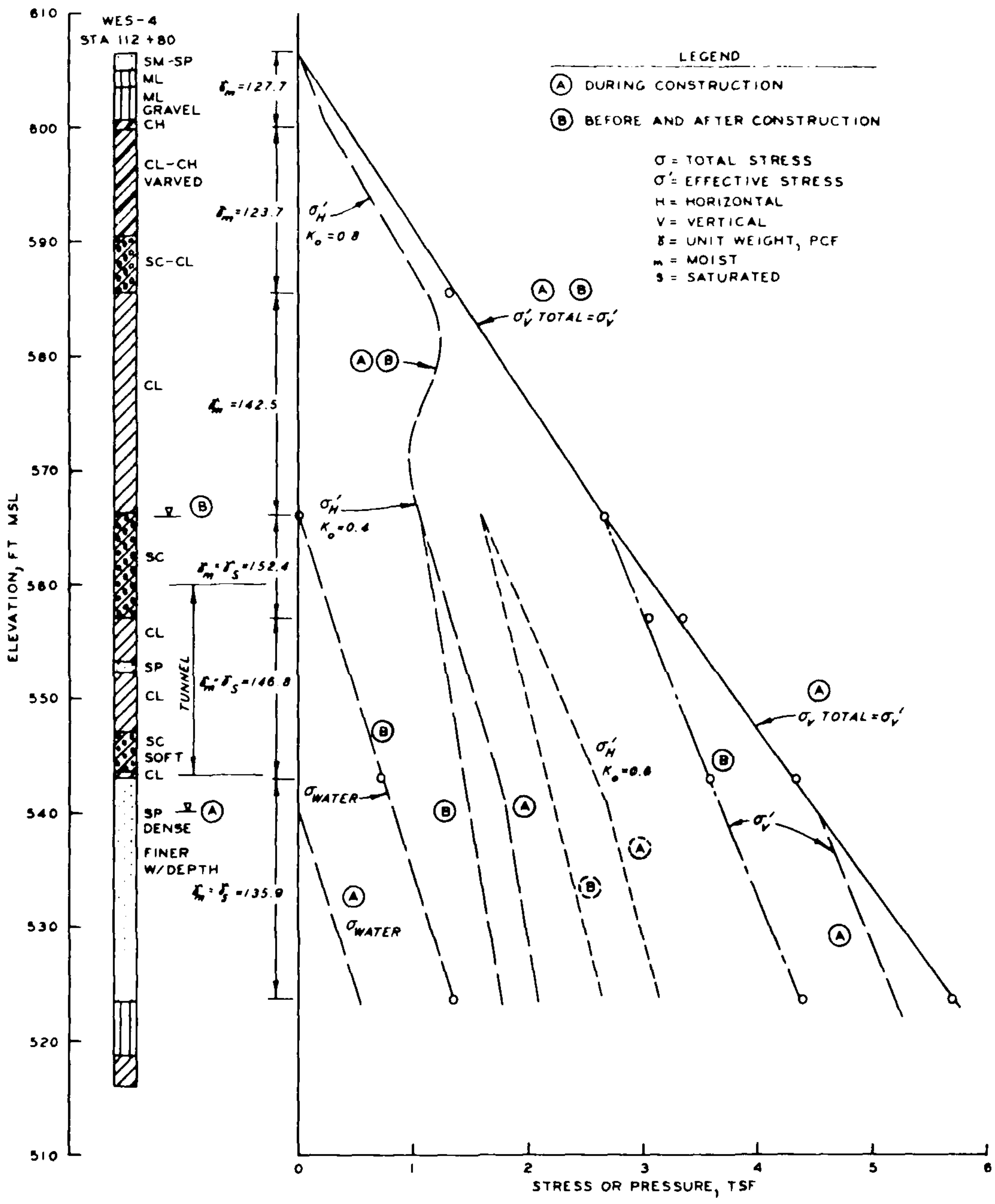


Figure 41. Estimated in situ soil stresses, Sta 112+80

soils, a total stress at rest coefficient (K) is used and determined (Dunlop, Duncan, and Seed, 1968) from:

$$K = K_o + (1 - K_o)\ (\gamma_w\ /\ \gamma_{sat})$$

where γ_w and γ_{sat} are the unit weight of water and of saturated soil, respectively. Listed below are values of K_o and corresponding K values used in calculating estimated horizontal pressures on soil boundaries for the structural analyses. Corresponding values of ν' and ν (effective and total Poisson's ratio, respectively) are also shown ($\nu = K/(1+K)$). These values change very little as shown in Figure 42 for the range of saturated unit weights (136 to 152 pcf) used to determine in situ stresses.

K_o	ν'	K	ν
0.40	0.29	0.67	0.40
0.60	0.38	0.78	0.44

Values of K and ν for total pressures are higher and have less variation than K_o and ν' values. Thus, K values of 0.7 and 0.8 were selected to cover the possible range for in situ conditions.

c. Modulus values. Modulus of elasticity (E) values needed in the structural analyses were based on those in Table 15 from $K_o\bar{R}$ undrained loading. Sample stress release affects laboratory measured values. Studies of E values from in situ field tests and laboratory tests show that field measured values for highly overconsolidated till are as much as twice the laboratory values (Radhakrishna and Klym, 1974). Consequently E values in Table 15 were increased by a factor of 1.5 to account for stress release and possible disturbance. The value recommended for the 60 ft clay and silt unit was 3000 psi. A value of 6000 psi was recommended for fine sands to sandy silts in the beach sand unit. These values also tend to agree with those in Table 16 for the Midland Nuclear Plant Site.

d. Effect of restored groundwater. As shown in Figures 40 and 41, restoration of the groundwater results in lower vertical and lateral effective stresses from the buoyancy effect of the water. However, the total

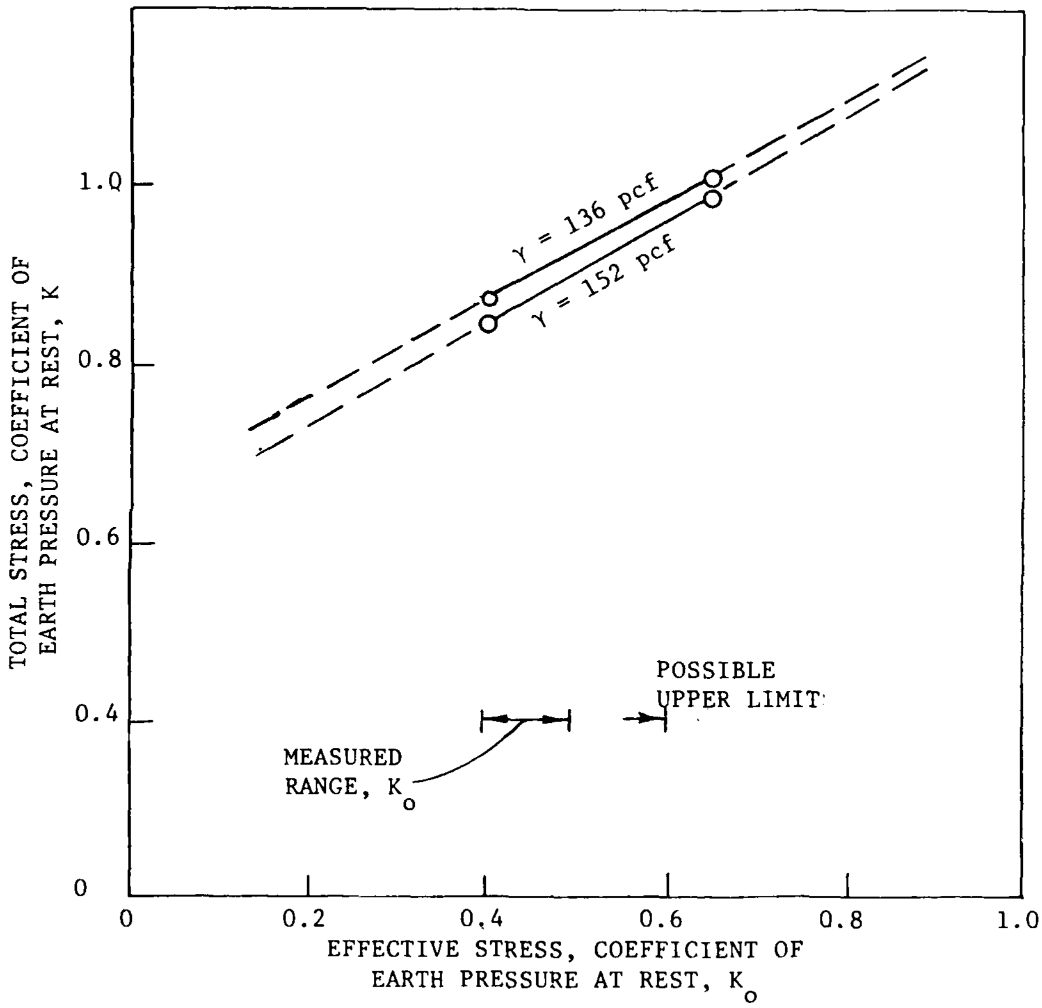


Figure 42. Range of at rest earth pressure coefficient K_o and K versus unit weight, 15 Mile Road/Edison Corridor Tunnel Failure Study

stresses in the soil below the restored water level are essentially the same as during dewatering since the total vertical and horizontal pressures include the water pressure. The total stress horizontal pressure is K times the total vertical pressure. Water pressures in the sands, silts, and clays increase linearly with depth below the restored groundwater level as shown in Figure 40 and 41. The sands and silts with little or no cohesion were found to be susceptible to piping (para 66) with the cohesionless sands being highly susceptible, under water pressures as low as 0.75 psi. The water pressures in the soils around the base of the tunnel would be equal to the effective water level above the tunnel at elevation 560 to 568 (as a minimum, Figure 21) less the water level in the tunnel at approximately elevation 550 (for a water depth equal to about one-fourth the inside diameter). This difference in elevation would correspond to an effective head of water of 10 to 18 ft or 4 to 8 psi. For soils around the tunnel but above the water level inside the tunnel, the water head would be equal to the difference between the soil elevations and effective water level elevations (minimum of about 8 ft or 3 psi at the springline). These water pressures are more than those required to cause piping. Consequently, material from fine sand and silty sand and silt strata supporting the sides and/or base of the tunnel would be piped into the tunnel through leaking joints. The supporting strata would loosen from loss of volume and a gradual loss of ground support would increase with time at the base and sides of the tunnel. Based on this series of events, the estimated in situ total stresses and soil properties for the following cases were provided for use in the structural analyses:

(1) Full overburden pressure (total vertical stress, σ_v) acting in the soil above the tunnel and horizontal pressures increasing with depth in the soil outside the tunnel of 0.7 to 0.8 times the vertical pressure (K times σ_v). These pressures would simulate conditions several years after construction when full overburden stresses would be developed over the tunnel.

(2) Using the above conditions, reduce the support at the bottom and then at the sides of the tunnel simulating piping and loss of support from pipeable strata found to occur along the tunnel base in all areas and along the tunnel sides in Distressed Areas 1, 2, and 3.

Structural

Concrete investigations

72. The following list of documents was used in preparing and evaluating the information and data contained in the Concrete Investigations portion of this report:

a. "Tests of Cement," Pittsburgh Testing Laboratory Report, Order No. 71956, Date: October 19, 1971, to the City of Detroit (Huron Portland Cement Type IA).

b. "Test Sample Identification Report," P. O. No. D16667, Date: September 15, 1971, Detroit Metropolitan Water Service, Engineering Division (Huron Portland Cement Type IA).

c. "Physical Portland Cement Test Results," T.E.C. Report No. 9275-8, Date: 3 November 1970, Testing Engineers and Consultants, Inc., Detroit, MI, for Detroit Metropolitan Water Services (Peerless Type I Portland Cement).

d. "Mechanical Analysis Report, Test No. 168, 6A Natural Aggregate," Metro Water Department-Detroit, Division of Engineering, Date: 15 July 1971.

e. "Mechanical Analysis Report, Test No. 169, 2NS Sand," Metro Water Department-Detroit, Division of Engineering, Date: 15 July 1971.

f. "Concrete Test Cylinder Reports," Detroit Metropolitan Water Service, DE 9187:

Report No.	Date
614	25 April 72
627	2 May 72
632	3 May 72
644	9 May 72
659	16 May 72
710	19 June 72
720	27 June 72

g. "Inspector's Report," City of Detroit, Department of Water Supply, Division of Engineering, Form D-144-BO-A for the period 15 April 1972 through 12 June 1972.

h. "Contract Documents for Oakland-Macomb Interceptor System, Corridor Interceptor - Section III, Division 2, Concrete," Detroit Metropolitan Water Services, Pollution Control Program, Contract No. PCI-7, September 1969.

i. Letter from Charles Beckham, Detroit Water and Sewerage Department to Lawrence Maloney, Detroit District, Corps

of Engineers, Date: 20 June 1980, regarding: Sewer Tunnel Failure - 15 Mile Road and Edison Corridor, Sterling Heights, Michigan.

j. Compilation of 163 color photographs taken between May and July 1980 of the entire tunnel from Sta 110+00 to 133+79.

73. Visual observation

a. Visits were made to the site by the WES team on 12-29 May 1980 and 19-20 June 1980. The first visit involved a visual inspection of the tunnel, photographic documentation, loose sample collection, tunnel deformation measurements, drilled core location selection, and the starting of the concrete drilling operation. The second visit was to ensure that all concrete sampling was satisfactorily completed before the drilling equipment was dismantled and removed from the tunnel. Upon entering the site, several piles of large concrete blocks and debris could be seen stored to the side. The blocks were approximately 1/2 cu yd in size and mostly irregular in shape. From close inspection of the debris, small holes had been drilled for easier removal of a portion of the tunnel concrete down to the springline for installation of the access shaft. The holes were approximately 2 in. in diameter and apparently drilled through the entire blocks. Each block had several holes spaced 5 or 6 in. apart.

b. Entrance into the Oakland Macomb Interceptor Sewer was by a 4-ft-diam manhole about 50 ft deep. In the tunnel the lighting was inadequate and visibility was poor. Hand-held flashlights had to be used to supplement the lighting. Inspection of the invert was impossible because of the sludge remaining in the sewer. Interest was concentrated on the crown and two springlines until the sludge could be removed.

c. Distressed Area No. 3, located between Stas 115+50 and 111+25, was the longest damaged area of three distressed areas. Area No. 3 had a series of longitudinal cracks along the crown, almost the entire length of the break area. The width of the longitudinal cracks was 1-1/2 in. at the widest point near Sta 114+00. These longitudinal crown cracks appeared to be fractured the entire depth of the concrete, with most cracks running perpendicular to the surface. This was confirmed as a steel rod was slid into the crack and the approximate angle was noted.

d. Branches of cracks diverged from the longitudinal cracks down to the springlines where they joined two rows of small fractures on both walls. The fractures appeared similar to those of a shear failure, where the fractures

were short and angular and primarily in a straight row.

e. Following the final clean-up of the sludge in Distressed Area No. 3, the invert was noted to have several longitudinal cracks along much of the break area. Along Sta 114+25, the longitudinal crack appeared to be uplifted some 3 in. This looked as though the invert was under pressure, finally cracked, and lifted up on one side. Seepage of groundwater was occurring through the cracks in the invert.

f. With construction joints located every 105 ft, the circumferential cracks that existed were easily distinguished from the normal construction joint. These circumferential cracks looked very similar to the construction joints in that the cracks were almost perfectly straight and smooth. Seepage of ground water could be seen coming in through the cracks that were as wide as 3/4 in.

g. At Sta 114+42 some material had filled a void near the springline in the concrete. The material was some type of silty sand. It appeared that the void was from construction and that the sand had seeped in to fill the void. With much of the material removed, the void was measured to a depth of at least 15 in.

h. Also at this station (Sta 114+42), the north end of a 105-ft section that began at Sta 113+37, the invert from the 5:00 o'clock to the 7:00 o'clock position had sunk 4-1/2 in. Water and sludge had begun to pool from the seepage.

i. Several locations in Area No. 3 had small cracks and crevices where groundwater was springing up about 1-1/2 in. This was caused from the water table being up near the springline. Water pumps were on continuously to keep the water in the tunnel to a minimum.

j. Moving upstream from Area No. 3 in the nondistressed area, lines representing the cold joints in the concrete could easily be seen. The concrete apparently flowed downstream or south as it was being placed from the angle of the cold joints. The WES drill crew took concrete core samples from a cold joint to determine whether it had any detrimental effects on the concrete. Also at this station (Sta 119+15), another core was taken from another cold joint that exhibited some efflorescence. Efflorescence is a deposit of salts, usually white, formed on the surface of concrete. For efflorescence to occur, water is needed to carry the salts contained in the concrete to the surface of the concrete, either by vapor transfer or by liquid diffusion. In liquid form, appreciable

quantities of soluble salts are deposited at or near the surface of the concrete when the water evaporates. If there are cracks or joints present, the water moves quickly along these surfaces carrying the salts with it. The core revealed a clay-like sludge just under the surface and extending back several inches through the core. Then the sludge appeared to extend perhaps the entire thickness of the concrete wall.

k. Of the three distressed areas, Area No. 2 was the only area where no concrete had fallen into the tunnel. Area No. 2 was highly fractured with longitudinal cracks in the crown and invert. Cracks at an angle of 50 to 60 deg with the horizontal were all along the springlines, extending up to the longitudinal cracks in the crown. Massive pieces of concrete appeared to just dangle in the crown and springlines. The invert had a few locations where the concrete sunk an inch or two. In a couple of circumferential cracks, efflorescence and water appeared seeping through the concrete.

l. Distressed Area No. 1 was the most damaged area. Here the massive pieces of concrete along with the steel ribbing and wood lagging had collapsed into the tunnel. Area No. 1 had concrete that was actually dangling from the crown. For safety reasons the new ribs and lags were installed in this area first, both on the north and south ends of the collapsed area. Concrete in Area No. 1 exhibited similar cracks as those in the previous areas--longitudinal cracks in the crown and invert, with angular cracks in the walls. There appeared to be more cracks running in other directions than before. The concrete did not exhibit any efflorescent deposit, nor was ground water seepage visible. The concrete in Area No. 1 had no rebars (or no rebars that were visible). The concrete thickness appeared to be adequate when observed.

m. Figure 43 shows a typical horizontal cold joint accompanied by slight honeycombing. This type of condition was prevalent throughout the tunnel except that most cold joints were inclined from 10 to 20 deg from the horizontal. Figure 44 shows another typical cold joint but with efflorescent deposits due to passage of liquid through the joint. Figure 45 shows a large inclusion of mud that either fell into the concrete during casting or that was extruded into a honeycomb void in the concrete. Table 17 is a summary of the concrete data.

74. Crack patterns

a. Photographic documentation (163 photographs) was made for the entire length of tunnel in question (Sta 110+00 to 113+79). These photographs show the extent of cracking

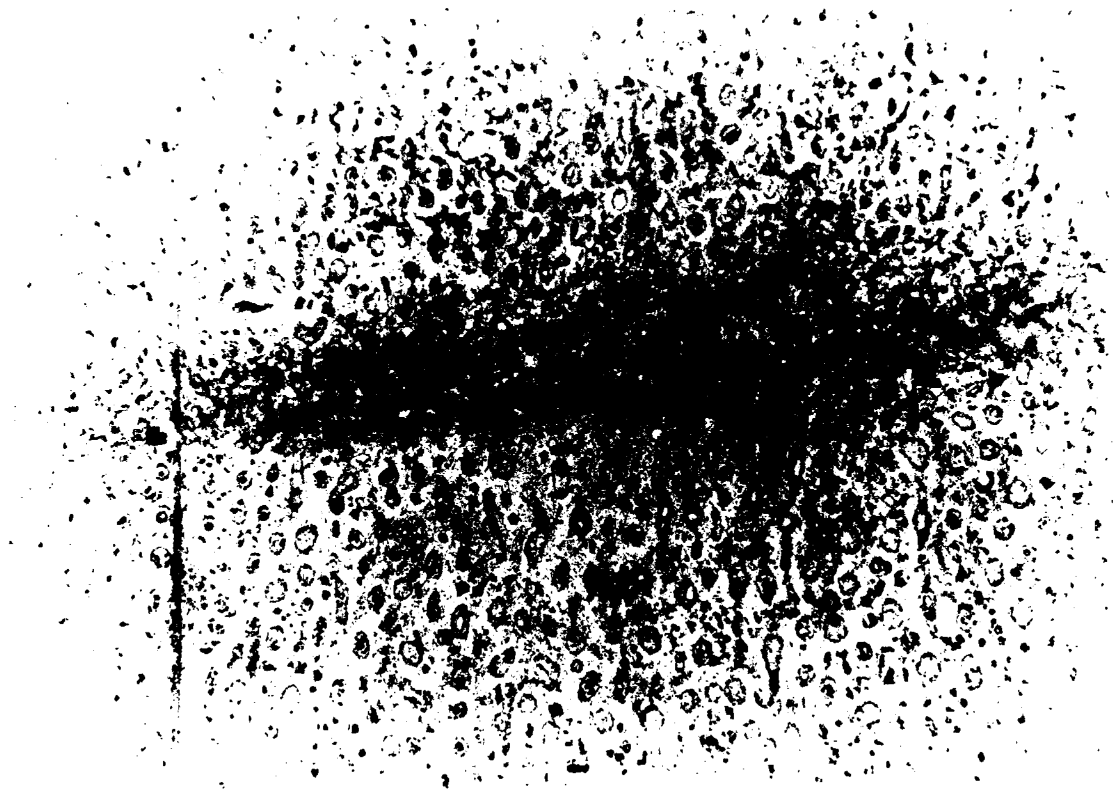


Figure 43. Typical horizontal cold joint and slight honeycombing seen on interior wall of culvert; this was not an area of failed concrete

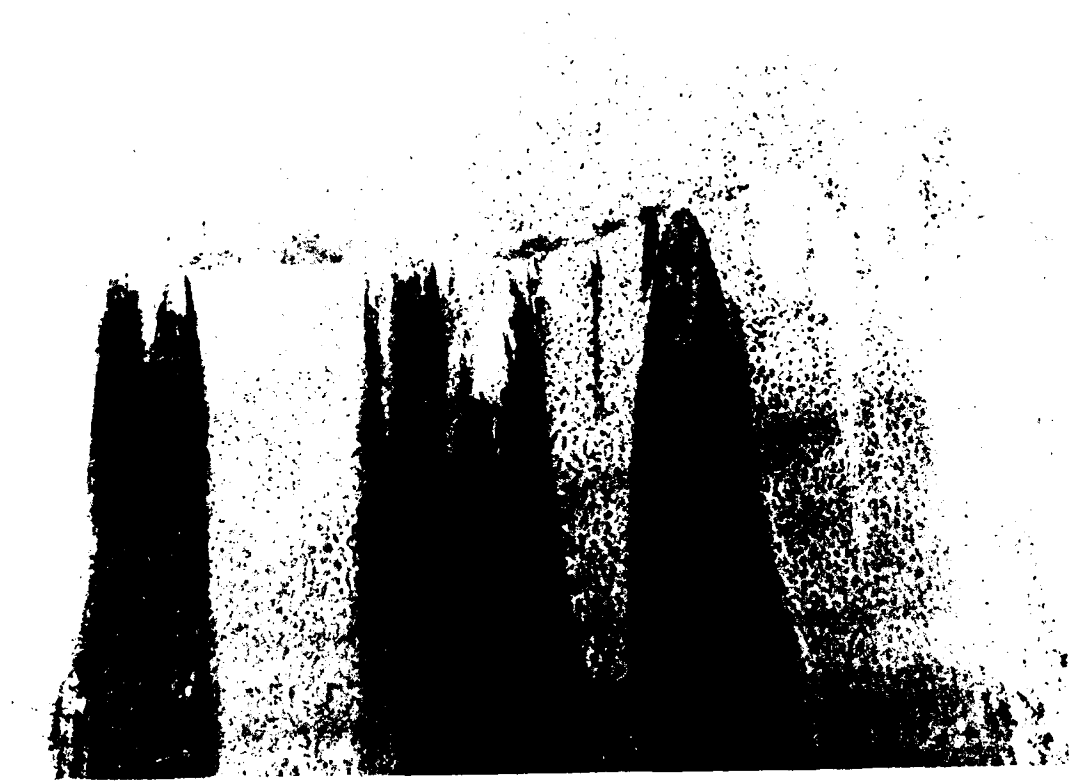


Figure 44. Cold joint on inner culvert wall in another nonfailed area; this joint shows efflorescent deposits due to passage of liquid through the joint

Figure 45. Inner surface of culvert wall at [illegible] Area 3 at Sta 114+42; the blob in the center of the [illegible] that fell into place between concrete lifts during the [illegible] was extruded into a honeycomb void; [illegible] least 15 in. deep into the [illegible]

that existed at any location during this study. In order to obtain a composite view of the cracks, an overlapping series of 35-mm photographs were taken of the fractured surface area between Stas 113+25 and 114+75. This 150-ft section approximates Area 3. The photographs, which were taken at the invert, water line, springline, center line, and crown at each location, were put together in an overlapping manner to form a one-dimensional mosaic of the tunnel. Because of the photographic technique used to make the photo mosaic, the magnification of pictures near the invert was greater than that of the pictures made near the crown. The cracks in the photographs were then sketched on paper and scaled down to a more manageable size. The resulting crack pattern for Area 3 is shown in Figure 46. Insufficient photographs at Areas 1 and 2 prevented a complete mosaic compilation in those areas, but the patterns are similar to Area 3. The perspective in viewing Figure 46 is that the round culvert is drawn as if it were flat. The true orientation of the cracks can be obtained by rolling a copy of Figure 46 into a tube around the longitudinal axis of the tunnel and placing the crown or invert in the proper location. Figure 46 shows all forms of cracking, i.e., longitudinal, circumferential, and diagonal. Evidence of crushing at the springline also can be seen. The water line indicated is actually the water mark that existed at the time of the survey. It was very pronounced due to the collection of solid matter left on the wall.

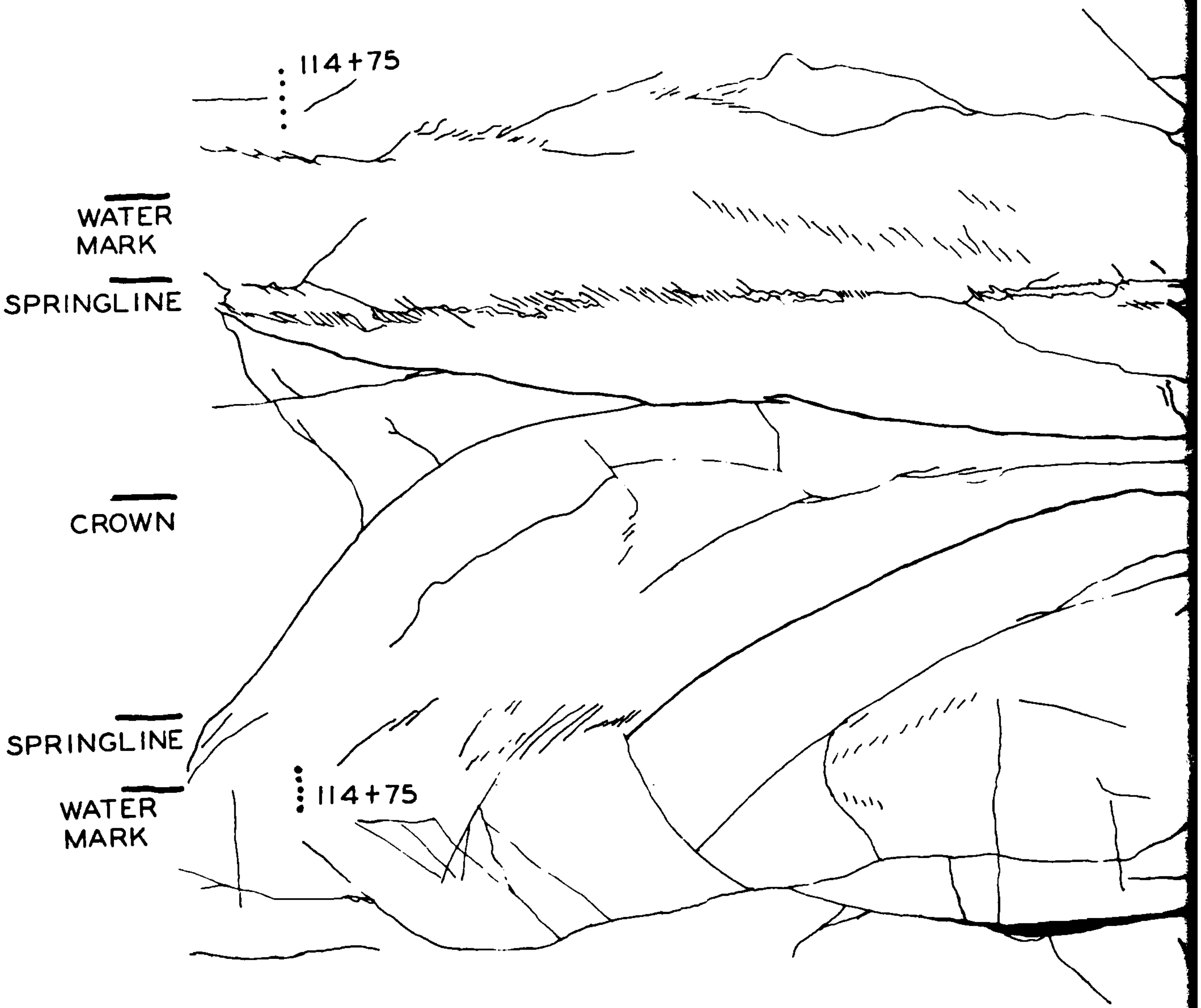
b. Cracks were also present in several of the drilled cores that were obtained from the tunnel. Figures 47, 48, 49, and 50 show some typical cracks in cores.

75. Nondestructive measurements

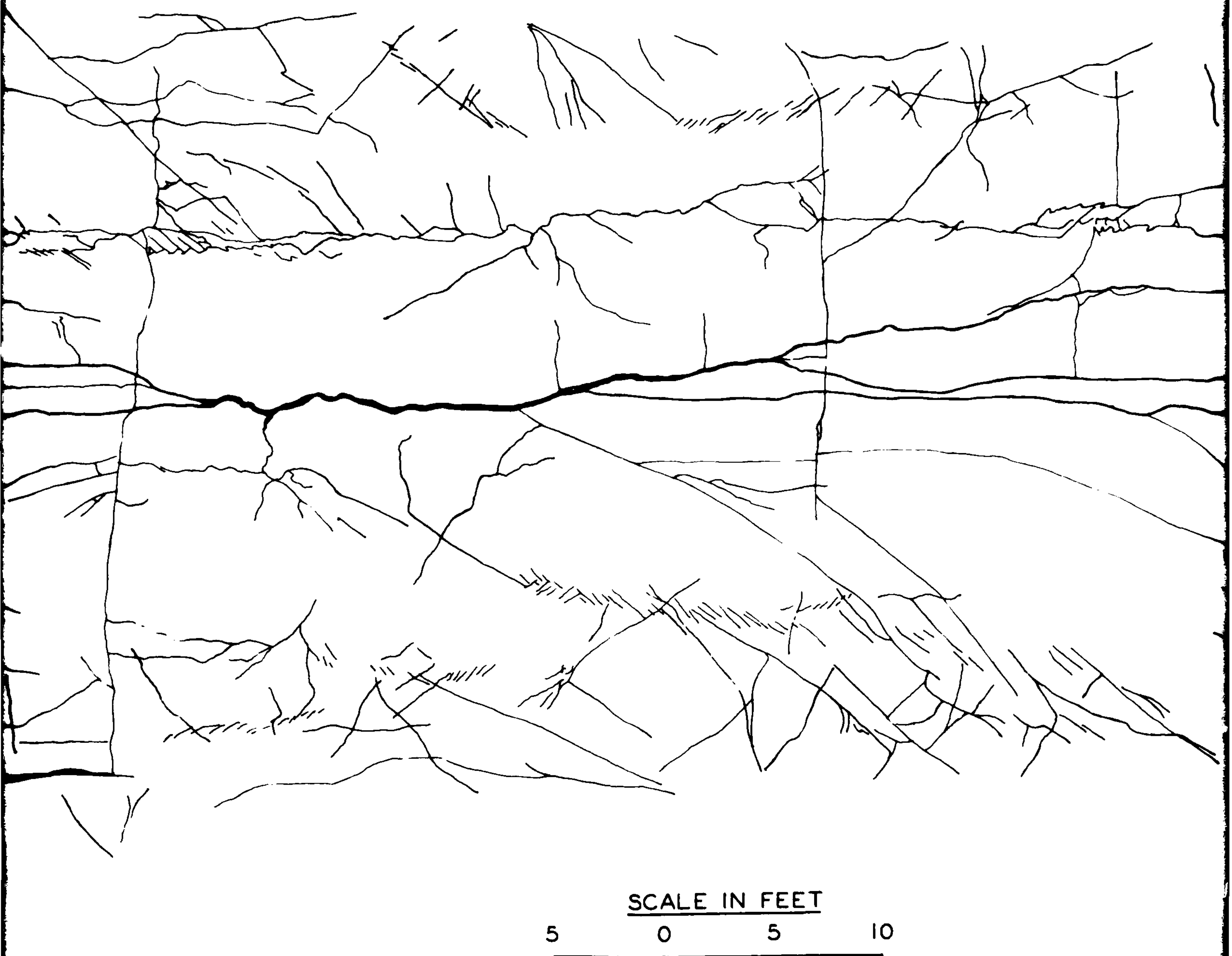
a. Nondestructive measurements were obtained from the collapsed tunnel using a testing apparatus called the Schmidt Concrete Test Hammer. These measurements are used to estimate concrete strength. The Schmidt Hammer is a calibrated spring-operated impact device that is triggered as it is pressed against any hardened concrete surfaces. As the plunger impacts against the concrete surface and rebounds back through the hammer, the spring measures the amount of impact exerted and displays a numerical "Rebound Number." Empirically the correlation between rebound number and strength of the concrete has been derived from a great number of hammer tests.

b. In the WES evaluation of the concrete, compression tests were performed on several cored concrete specimens taken from the sewer tunnel. Prior to compression tests, the Schmidt hammer was used on all the specimens. Rebound

Blank

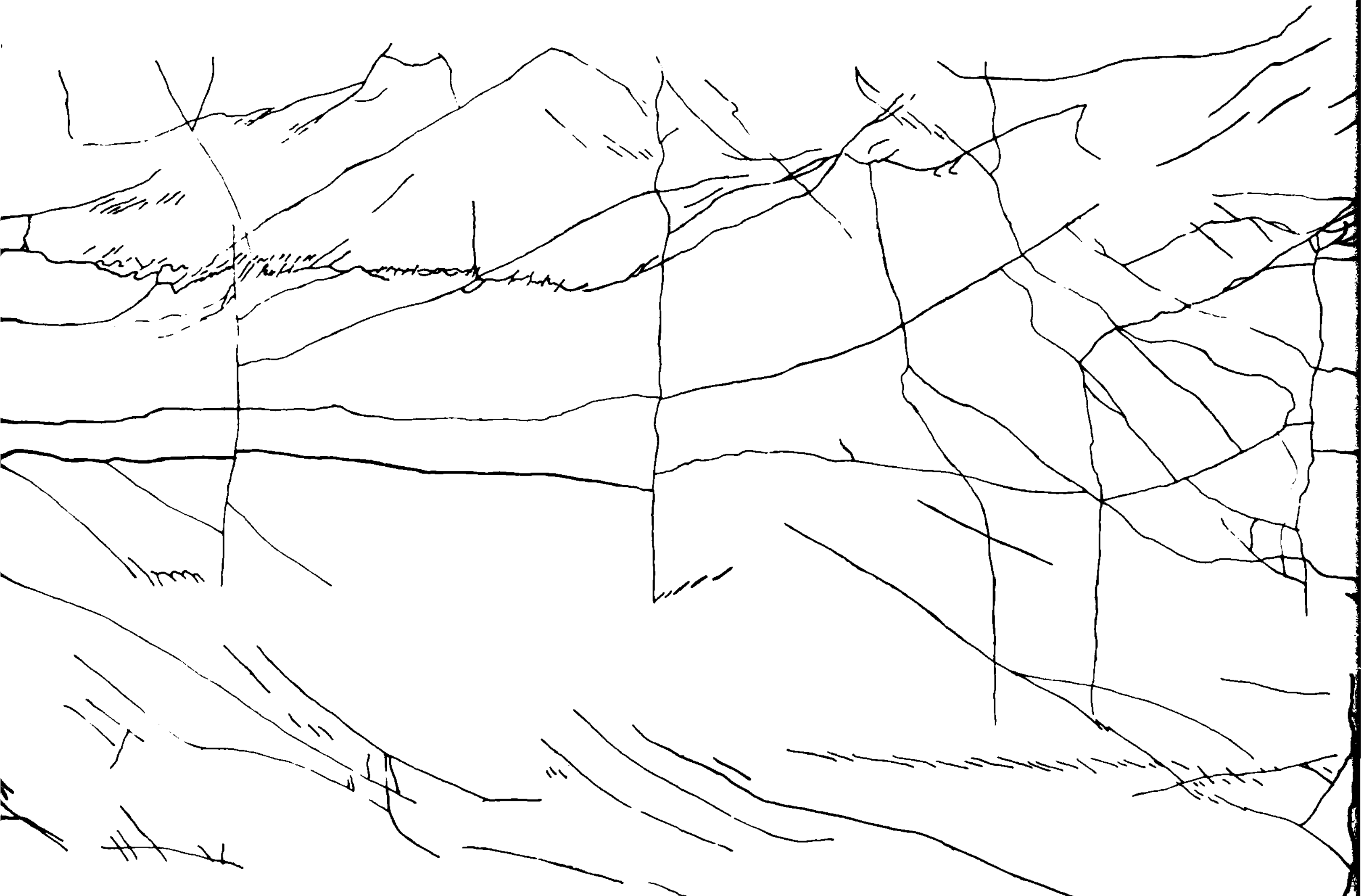


114+75
WATER MARK
SPRINGLINE
CROWN
SPRINGLINE
114+75
WATER MARK



2

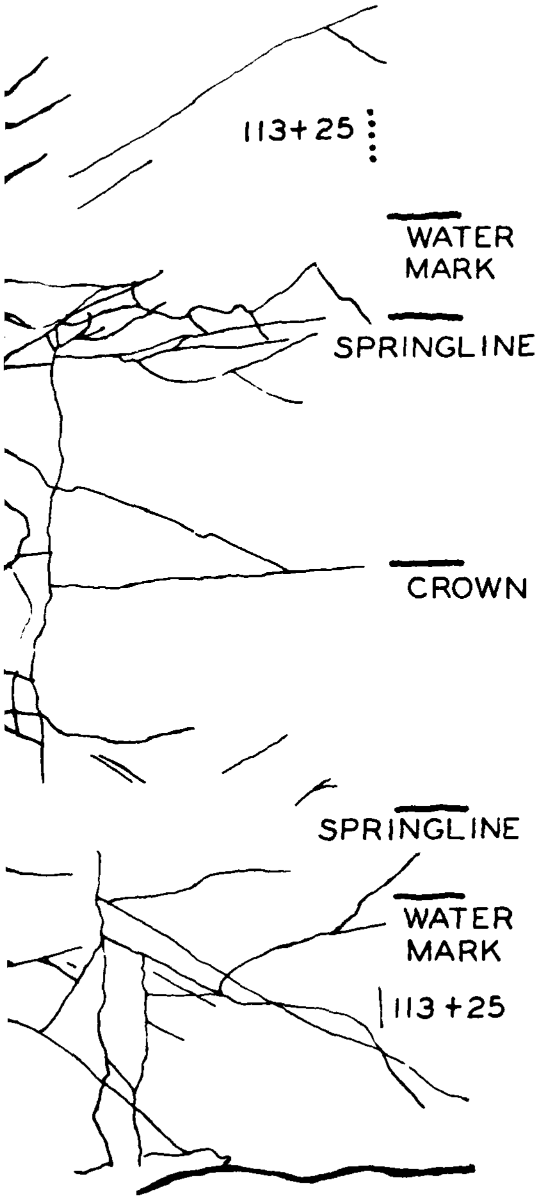
V



NOTE: NO EXACT SCALES CAN BE ASSIGNED BECAUSE OF SOME PHOTOGRAPHIC DISTORTION. THE POSITIONS OF THE CROWN, SPRINGLINE, AND WATER MARK WERE APPROXIMATED FROM THE PHOTOGRAPHS.

Figure 46.

3



46. Crack pattern, Distressed
Area 3

1 4

Figure 47. Six-in. diameter culvert core DET-4 CON-21 drilled over a crack at Sta 113+45 in Distressed Area 3

Figure 48. Above are DET-4 CON-21 crack surfaces opened along crack to show sediment coating

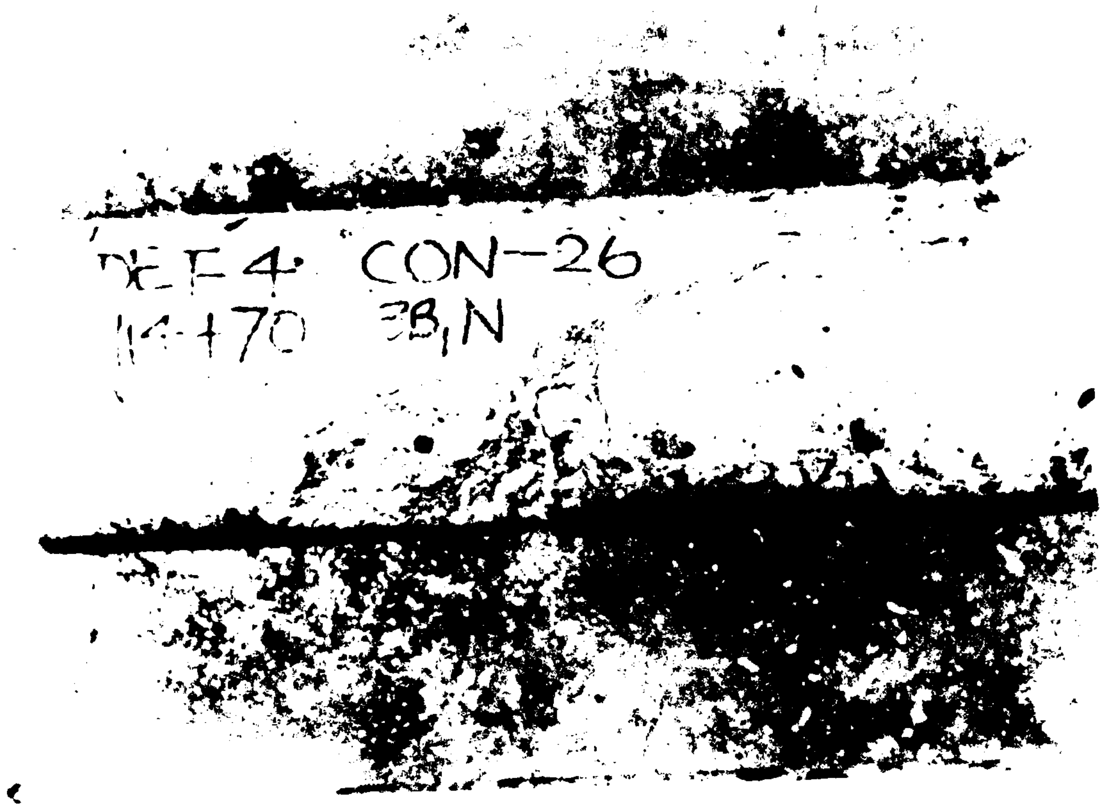


Figure 49. Six-in. diameter culvert floor core DET-4 CON-26 that intersected a crack, Sta 114+70, in Distressed Area 3

Figure 50. Above core DET-4 CON-26 opened on the crack; this crack was not coated with sediment; the powdery material is sawdust

numbers were taken on both the inside sewer surface and on the ends and the sides of twenty-four 6- by 12-in.-long cores. The results of the compressive tests ranged from 4440 to 7870 psi. The actual compressive strengths and the accompanying Schmidt hammer readings are shown in Table 18.

c. In obtaining the rebound numbers from inside the sewer tunnel, a 10-ft ladder or a special platform was needed to obtain readings taken from the crown and the two springlines. The curved tunnel surface and the slippery greasy sludge on the concrete surface made it difficult to reach the springlines and also the 12-ft-9-in. diameter of the tunnel made it difficult to test the concrete in the crown.

d. Rebound numbers were obtained at every station location that was involved in any other part of the concrete investigation. That is, when the core drilling crew took a concrete sample at a particular station, the Schmidt hammer was used to obtain rebound numbers from the crown, the invert, and the two springlines. Readings were also taken at all diameter change measurement stations. Four hammer readings were obtained from each of the four locations at each of the 35 stations evaluated by the WES (Table 19).

e. Measurements were made as close to specified locations as possible. Cracks, irregular surfaces, large coarse aggregates, and sludge hindered the measuring at several locations. In Distressed Area 1 the ribbing and lagging were being installed as a safety precaution and as part of the repair work. This caused most of the measurements at these particular stations to be taken off-center of the four locations.

f. The Schmidt hammer is designed to measure concrete that is in the vertical position with the hammer perpendicular to the surface or horizontal. Any deviations from the normal position, such as measurements taken from the crown and the invert, had to be corrected for inclination.

g. Detroit District personnel assisted the WES in obtaining the Schmidt hammer readings and also in obtaining a spare Schmidt hammer when the one from the WES was inoperative.

h. Rebound numbers obtained ranged from a low of 16 in the invert to a high of 48 in the crown. The rebound numbers suggest that for the most part the concrete in the crown and walls is fairly similar over the entire length of the sewer studied. The rebound numbers obtained in the invert are consistently lower, however, suggesting a

poorer quality of concrete in this location. The strength values for invert cores do not bear this out, however. Upon closer examination of the rebound numbers taken on the drilled cores from the invert, the following trend results:

Station	Rebound Number Core End	Rebound Number Core Side
132+50	29	37
127+50	38	33
117+25	34	39
114+70	30	43
113+45	28	34
112+18.5	31	41

With one exception (Station 127+50), the rebound numbers obtained from the surface that had been in contact with the sewage are consistently lower than the rebound numbers obtained from the portion of the core (center 12 in.) that was tested for compressive strength. As the rebound number is solely a measure of surface hardness, this suggests that there apparently was a softening or deterioration of the surface of the concrete in the invert but that once past the surface the concrete was similar to all the remaining concrete in the portion of the sewer studied.

i. In summary, the Schmidt hammer rebound numbers indicate that the quality of concrete between Stas 133+50 and 111+00 is fairly uniform. No significant deviations were observed.

76. Drilled core program

a. The objective of the drilled core program was to obtain representative samples of the concrete from all three break areas plus some cores from nondistressed areas and areas of special significance. In the break areas and nondistressed areas, cores were obtained from the crown, invert, and springline at each location. In general, care was taken not to include any visible cracks in the cores unless specifically desired. All cores were 6 in. in diameter. A total of 32 cores was obtained from the following locations:

Station No.	No. of Cores Taken	Locations and/or Purpose Represented
133+13	3	One each crown, invert, springline. Location north of break area 1. Minimally distressed.

Station No.	No. of Cores Taken	Locations and/or Purpose Represented
132+50	3	One each crown, invert, springline. Location just at beginning of break area 1.
131+84 (approx)	2	Two cores from a chunk of crown concrete excavated from that location in break area 1.
131+60 (approx)	2	Two cores from a chunk of crown concrete excavated from that location in break area 1.
127+50	3	One each crown, invert, springline. Location approximately center of break area 2.
119+15	2	Two cores. One drilled through an efflorescing cold joint; the other through a nonefflorescing cold joint.
118+35) 117+70)	2	One core at each location but representing both ends of the same concrete placement.
117+25	3	One each crown, invert, and springline in a nondistressed area.
115+47	1	One core taken through the construction joint.
114+70	4	One each at crown and springline. Initial core in invert contained a large crack and was redrilled next to the crack. Location approximately at north end of break area 3.
113+45	4	One each at crown and invert. Initial springline core was badly fractured and an additional core was drilled. Location approximately at center of break area 3.

Station No.	No. of Cores Taken	Location and/or Purpose Represented
112+20) 112+18.5)	3	One each at crown, invert, and springline. Location approximately at south end of break area 3.

b. All cores were drilled on the site except those from Sta 131+84 which were drilled at WES from a large piece of the crown concrete. The cores were crated and shipped to WES for evaluation.

77. Concrete thickness determinations

a. The requirements of the job called for a 16-in. liner thickness. The Detroit Water and Sewerage Department policy used on Contract PCI-7 allowed variations in thickness in those mined areas where it was considered impractical to remine to achieve required wall thickness and maintain plan alignment. In those cases a reduction in concrete wall thickness of up to a maximum of 25 percent was allowed providing the reduced concrete sections were reinforced with circumferential steel (#6 or #7 bars @ 6-in. centers) with longitudinal tie bars (#4 or #5 bars @ 12-in. centers). The reinforcing was to be placed such that it was embedded in the concrete at the inside face at the crown and invert and on the outside face at the springline. Verification of the concrete thickness was obtained by drilling both air hammer holes and cores out of the liner at selected locations. The concrete thicknesses measured in air hammer holes are shown in Table 11. The measurements made in drilled core holes are shown in Table 20.

b. The holes produced by the air hammer extended through the concrete and lagging into the surrounding soil. The thickness of the concrete was measured with a tape measure. No concrete was recovered from these holes. The thicknesses reported in Table 11 are only approximate as it was not always possible to drill perpendicular to the inside concrete surface. Variations from the perpendicular of 15 deg (which is probably the worst case) will only cause the measurements made to be 1/2 to 3/4 in. too long for the thicknesses of concrete involved. Attempts were made, however, to keep the air hammer as perpendicular to the inside surface as possible.

c. The drilling of the 6-in. concrete cores only extended to the lagging. The depth of the hole was again measured with a tape measure. As considerable care was used in aligning the drilling machine, the direction of drilling

was as close to perpendicular as possible, thus producing fairly representative concrete thickness determinations.

d. Of the 63 air hammer hole measurements and the 25 drilled core hole measurements, only four locations (six measurements) gave an indication of having less than 16-in. (approximate) thickness. These were:

Station	Method of Drilling	Location: Tunnel Side	Location: Clock hr North	Concrete Thickness ft
133+13	DC	Invert	6:00	1.2
133+13	DC	E. Wall	3:00	1.2
132+50	AH	E. Wall	4:00	1.25
132+50	AH	W. Wall	8:00	1.25
117+25	DC	Crown	12:00	1.1
113+45	DC	Crown	12:00	1.15

The core recovered at Sta 117+25 in the crown did contain reinforcing steel. No steel was observed in the other three drilled cores. The Inspector's Reports indicate that reinforcing steel was placed in the invert of the placement between Stas 133+32 to 132+27 and in the crown between Sta 117+57 to 116+52. The exact location of the steel with regard to station numbers is not clear from the reports, but the reports suggest that the locations did not include the entire length of the placement. No steel was included in the placement which contained Sta 113+45. The air hammer drilling did not permit material recovery, hence it was not possible to tell if reinforcing was present in these locations.

78. Physical tests. The concrete was evaluated for compressive strength, modulus of elasticity, and Poisson's ratio, as well as being examined petrographically to determine its constituents and any problems they may be experiencing. Each drilled core was examined and marked for cutting into a 2 to 1 length-to-diameter ratio to be used in the compressive strength test. After cutting, the ends of the cores were used for the petrographic examination. Not all cores were suitable for physical testing because of the cracks or other inclusions (waterstops) they contained. Only 24 of the 32 cores were tested in compression, but all were examined petrographically and all had Schmidt hammer readings taken on them. A summary of the compressive strength, modulus of

elasticity and Poisson's ratio results are shown in Table 21.

a. Compressive strength. Compressive strength tests for cores were done in accordance with CRD-C27 (ASTM C42), "Standard Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete." The core strength ranged from 4440 psi to 7870 psi with an average strength of 6600 psi for 24 cores. The drilled core concrete was approximately 8 years old when tested. A more meaningful presentation of the strength data is as follows:

105-ft Placement Station Nos.	No. of Drilled Cores	Average Core Comp. Strength psi	Single, As-Cast Cylinder Strength (age) psi (days)
133+32 - 132+27	3	7150	3450 (14)
132+27 - 131+22	4	7690	3150 (7)
128+07 - 127+02	3	7400	4000 (7)
119+67 - 118+62	1	7060	3640 (7)
118+62 - 117+57	2	6310	3450 (7)
117+57 - 116+52	3	6590	3890 (13)
115+47 - 114+42	3	5330	3060 (7)
114+42 - 113+37	2	6570	2920 (7)
112+32 - 111+27	2	5875	3420 (7)

Increases in strength with time were approximately 2300 to 4500 psi which is consistent with the expected strength given behavior of concrete cured in an essentially 100 percent relative humidity environment such as the submerged sewer provided.

b. Modulus of elasticity and Poisson's ratio

(1) The chord modulus of elasticity and Poisson's ratio under longitudinal compressive stress were determined for selected concrete cores in accordance with CRD-C19 (ASTM C469), "Standard Method of Test for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." The longitudinal and transverse strains on each core were determined using two sets of two each 4-in.-long electrical resistance strain gages bonded to the core. Load was applied at a constant rate using a 440 kip Universal testing machine. All cores were tested to failure. The results are shown in Table 21. These results were not meant to be indicators of concrete quality, but were needed in the structural analysis of the liner.

(2) The average modulus of elasticity for the 15 cores that were evaluated for this property was 4.97×10^6 psi. The range was from 4.05×10^6 psi to 6.50×10^6 psi.

The average Poisson's ratio was 0.20 with a range of 0.15 to 0.29. Both the modulus and Poisson's ratio average values are reasonable for concrete of this age and strength.

c. Petrographic examination. The ends or pieces of the following cores were sawed along their longitudinal axis:

Station	Tunnel Side	Clock hr North	Break Area
133+13	Invert	6:00	1
119+15	E. Wall	2:00	Nondistressed
117+25	E. Wall	3:00	Nondistressed
112+20	Crown	12:00	3

One sawed surface from each of these cores was ground smooth and then examined with a stereomicroscope. The stereomicroscope was also used to examine the old and/or fresh fracture surfaces on the other cores as well as the sawed surface on the end pieces of each core. Powder samples of various pieces of core were also prepared for use in X-ray diffraction analysis.

d. Concrete (general)

(1) The examination indicated that the concrete was not air-entrained. The test report on cement for this job dated 19 October 1971 indicates that Huron Portland Cement Type IA was being considered for use. As Type IA is an air-entraining cement, the cement in question either did not perform satisfactorily in this regard or was not the Type IA represented by the test report. Only Type I was specified, however. In either case, it should have no bearing on the problem at hand. There was no evidence of any chemical attack such as caused by sulfates or acid common to sewers.

(2) The concrete cores, with few exceptions, appeared to be well consolidated, homogeneous, and in good condition. The sawed and ground surfaces of cores from Stas 117+25 and 119+15 (nondistressed areas) shown in Figures 51 and 52, and Sta 133+13 (break area 1) shown in Figure 53 illustrate the appearance of the concrete. Visually, there is no significant difference in the appearance of the concrete in cores from either the distressed or non-distressed areas of the liner. One notable exception occurred in a piece of concrete recovered on 22 August 1980 by the Detroit District personnel during the

Figure 51. Sawed surface of core DET-4 CON-15 from sand area as shown in Figure 55; it did not show efflorescence

Figure 52. Cut and ground surface of piece of 6-in.-diam culvert wall core DET-4 CON-10, Sta 117+25, a nondistressed area

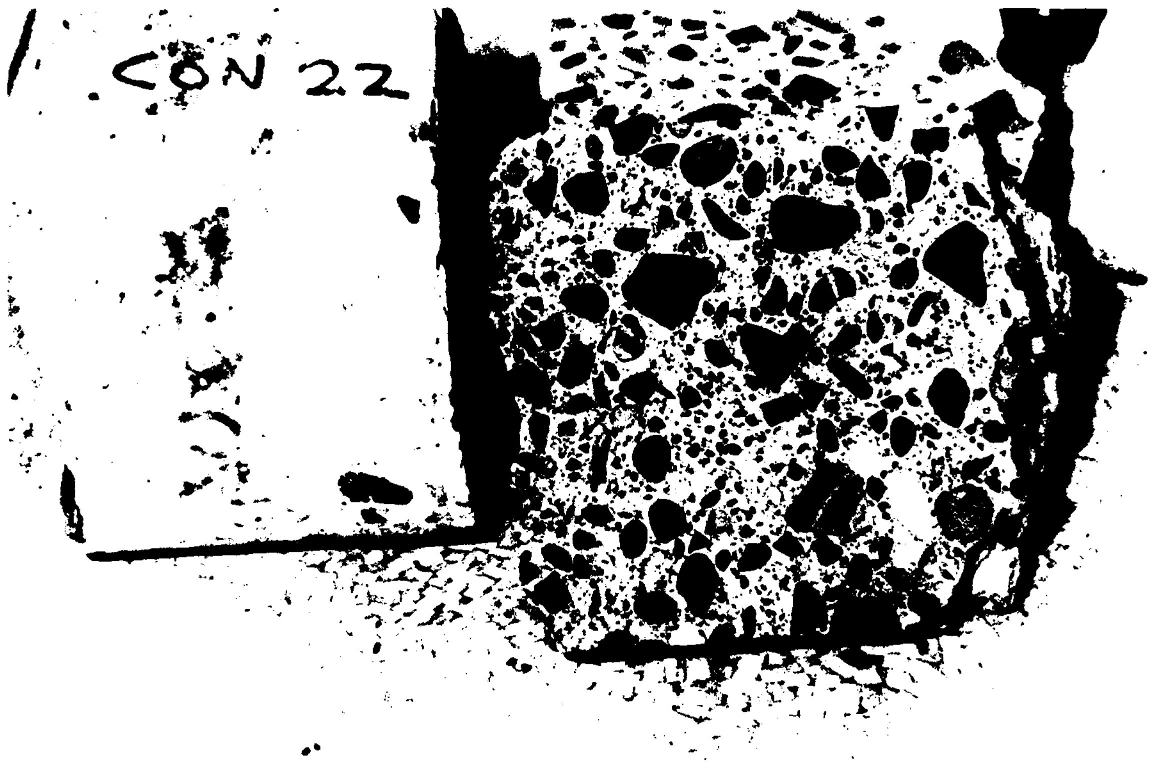


Figure 53. Similar surface as Figure 26, culvert floor core DET-4 CON-22, Sta 133+13, in Distressed Area 1

cleanout of the collapsed material from Distressed Area No. 1. The concrete came from the topmost quarter at Sta 131+48 and contained seams of a soil-like material. The seams began at the inside surface of the concrete forming an arc back into the concrete and then reappeared on the inside surface some distance away from their origin. The maximum depth of arc into the concrete was 5 to 10 in. Three seams of 1/4-in. thickness parallel each other approximately 1/2 in. apart. Aggregate was not visible near the seams. The concrete adjoining the seams appeared to have an excessive amount of air voids in it. A microscopic examination of the air void system showed that the voids were due to a lack of consolidation and that inadequate paste-aggregate contact resulted. The seam material was found to be sediment consisting of several nonclay minerals and minor clay minerals.

e. Aggregate. The examination also indicated that the coarse aggregate was 3/4-in. maximum size natural gravel made up of sedimentary, igneous, and to a lesser extent metamorphic rocks. The sedimentary rocks were primarily limestone and chert. The fine aggregate rocks were of the same composition as the coarse aggregate. There was no evidence of any deleterious chemical reactions between the aggregate and other constituents such as alkali-silica reaction. Some of the carbonate rock particles contained rims (Figures 52 and 53), but these were probably due to weathering before they were incorporated into the concrete as aggregate.

f. Cement

(1) Cement paste concentrates were prepared from the following cores:

Station	Tunnel Side	Clock hr North	Break Area
132+50	Crown	12:00	1
132+50	Invert	6:00	1
119+15	E. Wall	2:00	Nondistressed
117+25	Invert	6:00	Nondistressed
112+20	Crown	12:00	3
112+18.5	Invert	6:00	3

The paste was concentrated by gentle crushing of the concrete and then passing the fragments over a 150-mm (No. 100) sieve and then a 45-mm (No. 325) sieve. The material passing the 45-mm sieve was then packed in an appropriate slide mount and examined by X-ray diffraction,

using nickel-filtered copper radiation.

(2) Comparison of the X-ray diffraction patterns of the paste concentrate from each core showed that all six samples contained the same crystalline paste constituents; i.e., tetracalcium aluminate carbonate-II-hydrate (monocarbo-aluminate), calcium hydroxide, and ettringite. These are all normal paste constituents. The calcium silicate hydrate which is the amorphous "glue" that binds the constituents together was present in all samples, but is not readily detected by X-ray diffraction.

(3) Some contamination of the cement paste concentrate by aggregate substances was also present in the diffraction patterns. This included quartz, calcite, dolomite, and plagioclase feldspar. Amphibole, potassium feldspar, and kaolinite were also present.

(4) Since all of the paste examined was similar and normal in composition, and represented both concrete from distressed and nondistressed locations, it can be reasonably concluded that paste composition was not a significant factor in the cause of the problem.

g. Waterstop

(1) The Detailed Specifications, Section 2.II.9 Waterstop, allow the waterstop to be either metal (with coating), rubber or vinyl. Section 2.III.8c also requires that a waterstop, in addition to the joint coatings, shall be placed in each construction joint, where called for on the drawing, to produce water tightness. The Inspector's Reports indicate that, for the area of interest, waterstops were installed in construction joints at the following locations:

Station 111+27	Station 115+47
Station 112+32	Station 116+52
Station 113+37	Station 117+57
Station 114+42	Station 118+62

Using an electric drill and geologist hammer, the WES drill crew verified the waterstop presence by exposing it at Stas 111+27, 112+32, 113+33, and 114+42. A drilled concrete core at Sta 115+47 intersected the waterstop at that location and was included in the recovered core (Figure 54). No attempt was made to verify the waterstop at Stas 116+52, 117_57, and 118+62 as those locations were not exihibiting any apparent distress.

(2) The waterstop recovered from Sta 115+47 was a 6-in. corrugated (center bulb) polyvinyl chloride (PVC) made by Vinylex in Knoxville, Tenn. The piece was evaluated for

Figure 54. Six-in. diameter culvert wall core DET-4 CON-24 drilled at Sta 115+45; core was drilled through a plastic waterstop at a construction joint; note wood block at end of core (outer culvert surface) and color change of concrete at construction joint; this core was in Distressed Area 3

tensile strength and elongation in accordance with ASTM D412-68 (CRD-C573-74), "Standard Method of Tension Testing of Vulcanized Rubber," and CRD-C572-74, "Corps of Engineers Specifications for Polyvinylchloride Waterstop," with the following results:

Test	Specimen No.	Requirement	Results Obtained
Tensile strength psi (CRD-C573)	1	Not less than 1750 psi	2185 psi
	2		2350 psi
	3		2295 psi
	4		2175 psi
	5		2285 psi
		Average	2285 psi
Ultimate Elongation, % (CRD-C573)	1	Not less than 300 %	330 %
	2		320 %
	3		340 %
	4		310 %
	5		330 %
		Average	330 %

This piece of waterstop appears to still have satisfactory material properties after 8 years in service.

(3) The waterstop location verification procedures could not give any indication of whether or not the waterstop had been properly installed and was functioning properly. The recovered piece was also not much help in this regard. The waterstops observed did appear to be in the proper location in the concrete and had the proper amount of embedment. Both of these factors are not indicators of watertightness, however.

h. Other materials

(1) Samples of sediment which were found in the concrete and as coatings on fractured surfaces were ground and examined by X-ray diffraction. Some samples were also heat treated.

(2) Small samples of sediment from a void in the concrete at Sta 114+42 (Figure 45) and from the invert floor of the tunnel at Sta 120+71 were weighed and then heated at 110 deg C for one hour to remove free moisture. The samples were then weighed repeatedly until they reached a constant weight. They were then heated at 375 deg C for 16 hours and a final weight obtained. The weight differences were used to determine how much organic material was in them. Both samples were essentially free of organic matter, thus indicating they were not an accumulation of sewage.

(3) Portions of the samples noted in the previous paragraph along with material from a sediment inclusion in a core from Sta 119+15 (Figure 55) and sediment coatings on crack surfaces contained in the core from Sta 113+45 (Figures 47-48) were examined by X-ray diffraction. They all contained chlorite and clay-mica,* plagioclase feldspar, quartz, dolomite, and calcite. There were traces of vermiculite and kaolinite. The samples from Stas 114+40 and 120+71 also contained a little potassium feldspar, amphibole, and magnetite. The sample from Sta 114+40 contained a trace of gypsum. In general, all four samples were similar in composition and can be considered as a soil. Some of this soil had been included in the concrete during construction (Figures 54-55) and some had entered through cracks in the concrete (Figures 47-48).

(4) A sample of a stalactite from the crown of the tunnel between Stas 114+25 and 114+50 was ground and examined by X-ray diffraction. It was found to be composed of calcite plus a little clay and possibly some quartz. The occurrence and composition of the stalactite were not considered significant other than its presence demonstrated that water had passed through the concrete at that point. This passage of water had also been indicated by deposits or accumulations on the tunnel walls (efflorescence and rust stains) and by accumulations of soil in open cracks in other locations in the tunnel.

79. Diameter change measurements

a. The physical shape of the sewer tunnel was another phase of the investigation into the collapse of the tunnel. From the visual observations of the tunnel, it was noted that the tunnel was no longer circular but was elliptical in shape, particularly in the distressed areas. Deviations in the diameters of the cross sections from the nominal 12 ft 9 in. would give an indication of the magnitude of distortion.

b. A device to measure the inside diameters of the tunnel was designed and fabricated by Detroit District. This device consisted of a surveyor's leveling rod, a circular protractor, a measuring arm, and a pointer. The instrument was constructed around a central element, the surveyor's leveling rod which served as the stationary pivotal point. When the rod was extended from tunnel side to tunnel side and tightened, the pivot point was fixed. The protractor, graduated from 0 deg to 360 deg in 22.5-deg increments, was centered at the pivot point

* Clay-sized mica or illite clay or both.

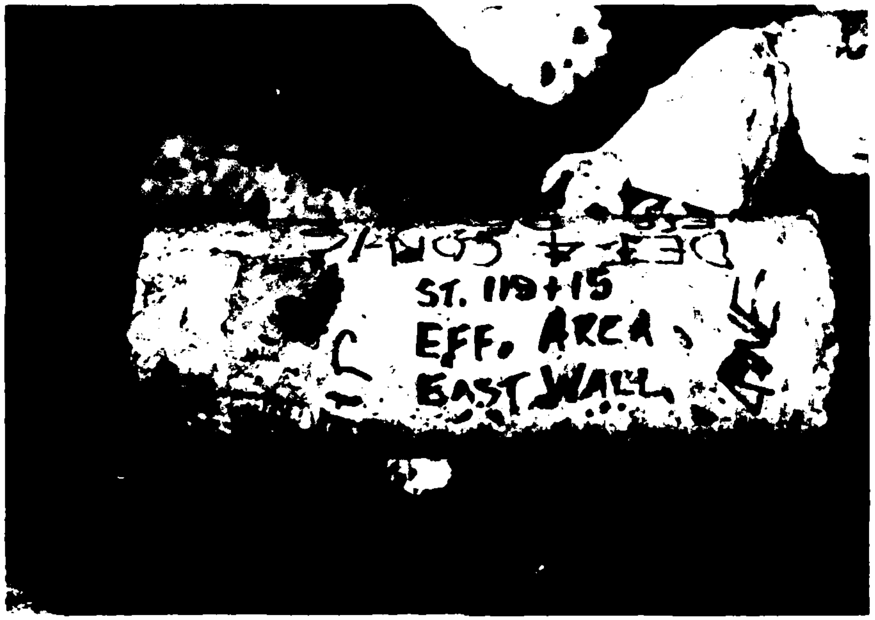


Figure 55. Six-in. diameter culvert wall core DEH-4 CON-16; it was drilled over the cold joint where it showed efflorescence and a sediment inclusion; Sta 119+15 nondistressed area

and zeroed with a surveyor's plumb bob. It was at these 16 angles that diameter measurements were taken for each station. The measuring arm consisted of a metal sleeve of fixed length of 5.66 ft and a 3-ft rod calibrated and marked at 0.01-ft increments. A pointer was attached to locate each angle. The entire instrument when in position rested on the shelf of a 6-ft stepladder.

c. Diameter measurements were taken at several critical stations along the entire tunnel, both in distressed and nondistressed areas. This was accomplished by positioning the ladder so that the edge of the shelf was at a particular station. The instrument was placed on the shelf and extended until both ends of the leveling rod were flush against the tunnel wall, then tightened. The protractor was positioned with the 180-deg angle toward the invert, the 270-deg angle to the west, the 0-deg angle toward the crown, and the 90-deg angle to the east. Any deviations from this position was noted and later corrected to the standard position. A plumb bob was used to zero the protractor with respect to the vertical.

d. The 5.66-ft sleeve of the measuring arm slid up and down along the calibrated 3-ft rod giving a maximum reading of 8.66 ft with a minimum reading of 5.66 ft. Readings were made by sliding the sleeve out until it was flush with the tunnel wall and recording the length of extension of the sleeve. The length of the extension plus the 5.66-ft sleeve equaled the distance from the fixed point to the tunnel wall. With the 16 measurements, the shape of the cross section of the tunnel was established at each station.

e. Two stations in Distressed Area No. 1, where severe cracking had occurred and where repairs had proceeded with placement of the ribbing and wooden lags, were measured using a shorter sleeve, 3.84 ft long, for accessibility over the ribs and lags that were being installed. With the shorter sleeve, several readings were made using the 3-ft rod and a cloth measuring tape to compensate for the shorter length.

f. The shape of the tunnel was graphically expressed by plotting the fixed point and each of the 16 measurements at their respective angles. Constructing lines connecting two points with the center point conveys the actual diameter of the tunnel. Any difference from the nominal 12-ft-9-in. diameter indicates the magnitude of the distortion. The results are shown in Appendix F.

g. The analysis of the diameter change measurement will be discussed further under the section Structural Analysis.

Summary of concrete investigations

80. The results of the concrete investigations are summarized as follows:

a. The coarse aggregate was a 3/4-in. maximum size gravel composed of sedimentary, igneous, and metamorphic rocks. The fine aggregate was composed of the same material. There was no indications that the aggregate was a contributing factor to the problem.

b. The cement paste was normal in composition and similar in all areas examined. The cement paste should not have been a significant factor in the cause of the problem.

c. The concrete was generally homogeneous and its quality was similar whether it came from distressed or nondistressed locations. Exceptions were the presence of continuous cold joints along both sides of the sewer. In these locations honeycombing, mud contamination, and seepage occurred. The concrete did not appear to be air entrained.

d. There was no evidence of any chemical attack on the concrete although rebound measurements on the sewer floor suggest a slight degradation of the surface skin of the concrete due to prolonged contact with the flowing sewage.

e. Cracks were present in the concrete, especially in the distressed areas. The presence of sediment coatings on some of the cracks indicates that the cracks were preexisting before the concrete cores were taken. It is impossible to tell, however, the age of the cracks. Efflorescence around the cracks and in the form of stalactites was composed of calcium carbonate and indicated that water and, in some cases, soil had been leaking through the cracks.

f. Most of the joints in the concrete appeared to be intact although leaking. Those leaking joints generally did not contain waterstops. The sediment associated with the leakage appears to be soil from outside the sewer. Some joints did contain waterstops. A recovered piece of waterstop indicated that the desired physical characteristics of the waterstop were still functional.

Mechanics of the structural failure

81. In this portion of the report, the mechanics of the structural failure of the sewer tunnel are discussed. This discussion begins with a description of the bypassed section of the interceptor

following its failure. Then, the loading assumed in the design of this structure is reviewed. Next, the resistance against this and other loadings implicitly required by the contract specifications are considered. Subsequently, based on the previously described results of the investigation of concrete material properties within the failed tunnel, the structural resistance available in the sewer tunnel is analyzed. Finally, using the previously discussed results of the geotechnical investigation of soil properties surrounding the sewer, the loadings imposed on this structure are investigated.

Description of structural failure

82. Figures 56 through 59 are photographs of a nondistressed and the three distressed areas of the tunnel which were taken in May 1980. In contrast to the nondistressed areas, the distressed areas are characterized by cracks and spalls running generally along the longitudinal direction of the tunnel for large distances compared with the diameter of the tunnel. Although some deviations from this pattern occur at circumferential construction joints and at the transition between distressed and nondistressed regions, these photographs suggest that the essential features of this structural failure should be explained by considering a representative transverse plane cross section of the tunnel perpendicular to its longitudinal axis. The photographs also show the cracking to occur mainly near the crown or the invert of the tunnel and the spalling to occur generally near the springline.

83. The deformation of the sewer tunnel along its length following its failure is depicted in Figure 60. The figure shows the difference between the structure's nominal 12-ft-9-in. inner diameter and the crown-invert diameter as a fraction of the nominal diameter. The points indicated by asterisks in this figure are based on a survey of crown and invert elevations performed for the Detroit Water and Sewage Department by the firm of Hubbell, Roth, and Clark in May 1980. It is notable that the crown-invert diameter was compressed to a value as much as 8 percent less than the nominal diameter in the distressed areas of the bypassed section. The consistent 2 percent compression of the crown-invert diameter in the nondistressed areas is probably

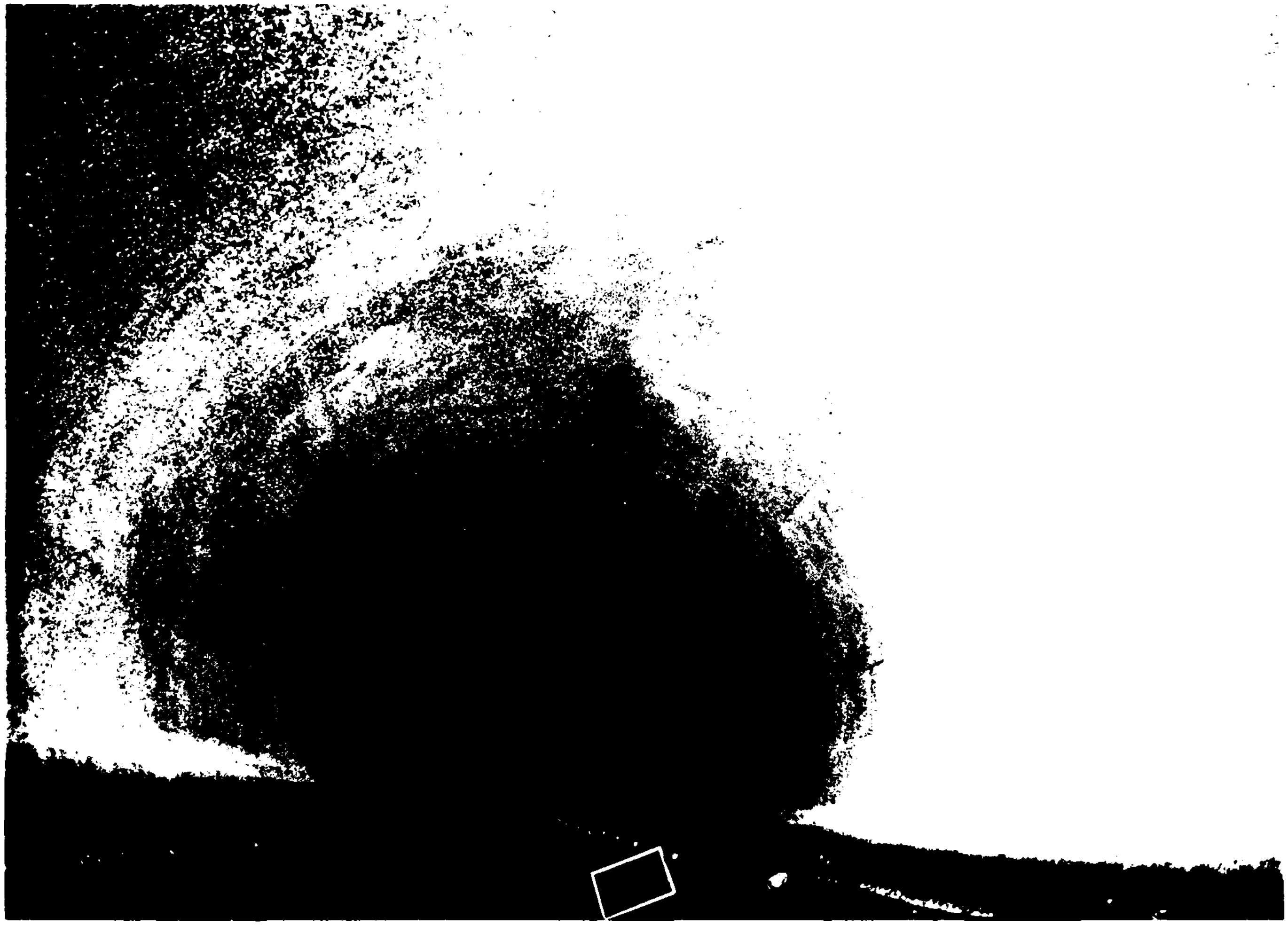


Figure [illegible]. Detroit Sanitary Tunnel [illegible]-7, Sta. [illegible] [illegible]

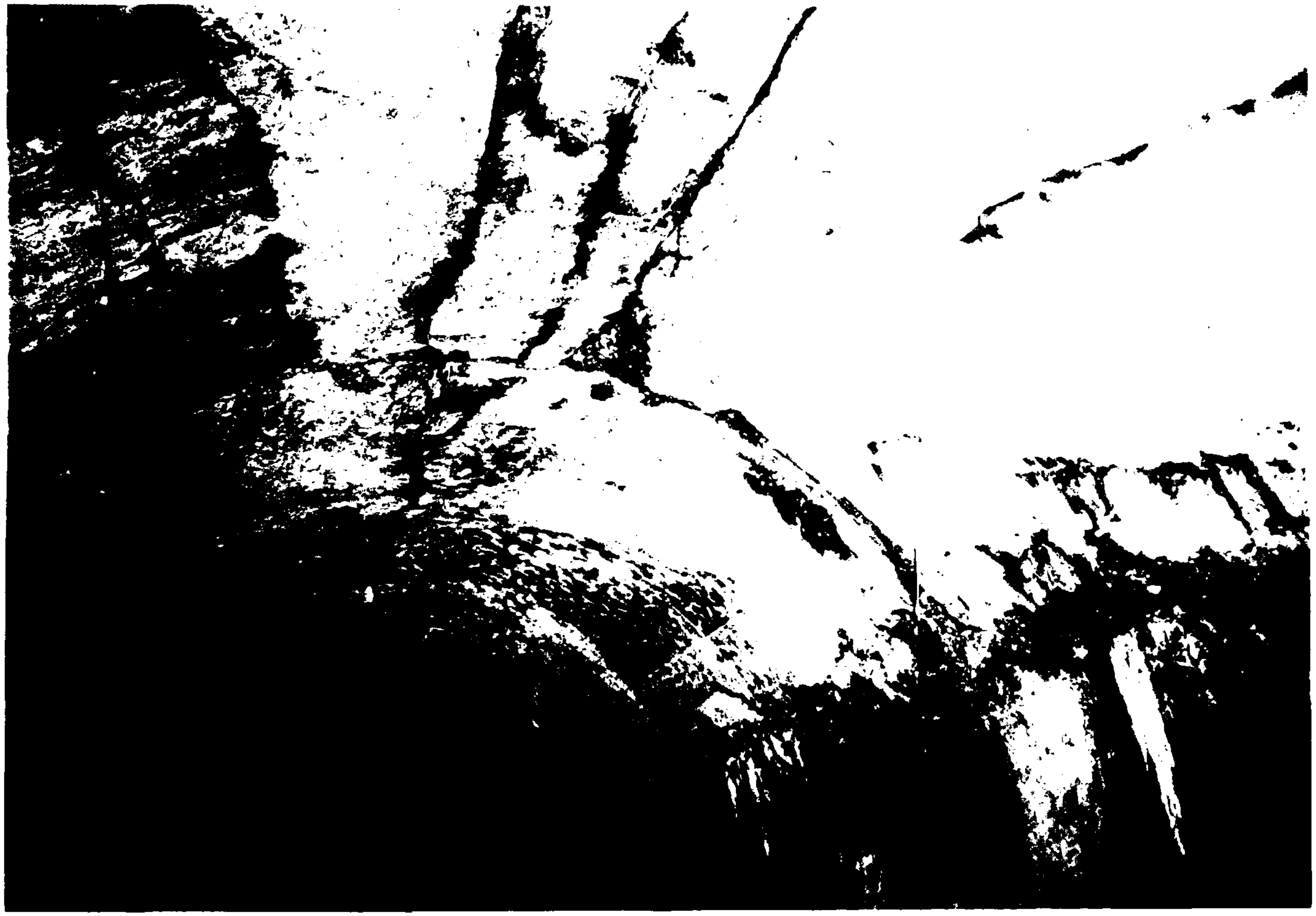


Figure 57. Detroit Sanitary Tunnel PCI-7, Sta: 114+50 Photo: 50 (19 May 80) South; long, trans & [illegible] fracs C & W wall; [illegible] 11[illegible]+[illegible]

Figure 58. Detroit Sanitary Tunnel [illegible], Sta: 107+50 Photo: [illegible] North; displaced long tunnel front [illegible] concrete

Figure 59. Detroit Sanitary Tunnel PCI-[illegible], Sta: 131+50 Photo: 136 ([illegible] May [illegible]) North; close up of south end break area #1; settlement pin in cavity; ribbing & lagging; [illegible]

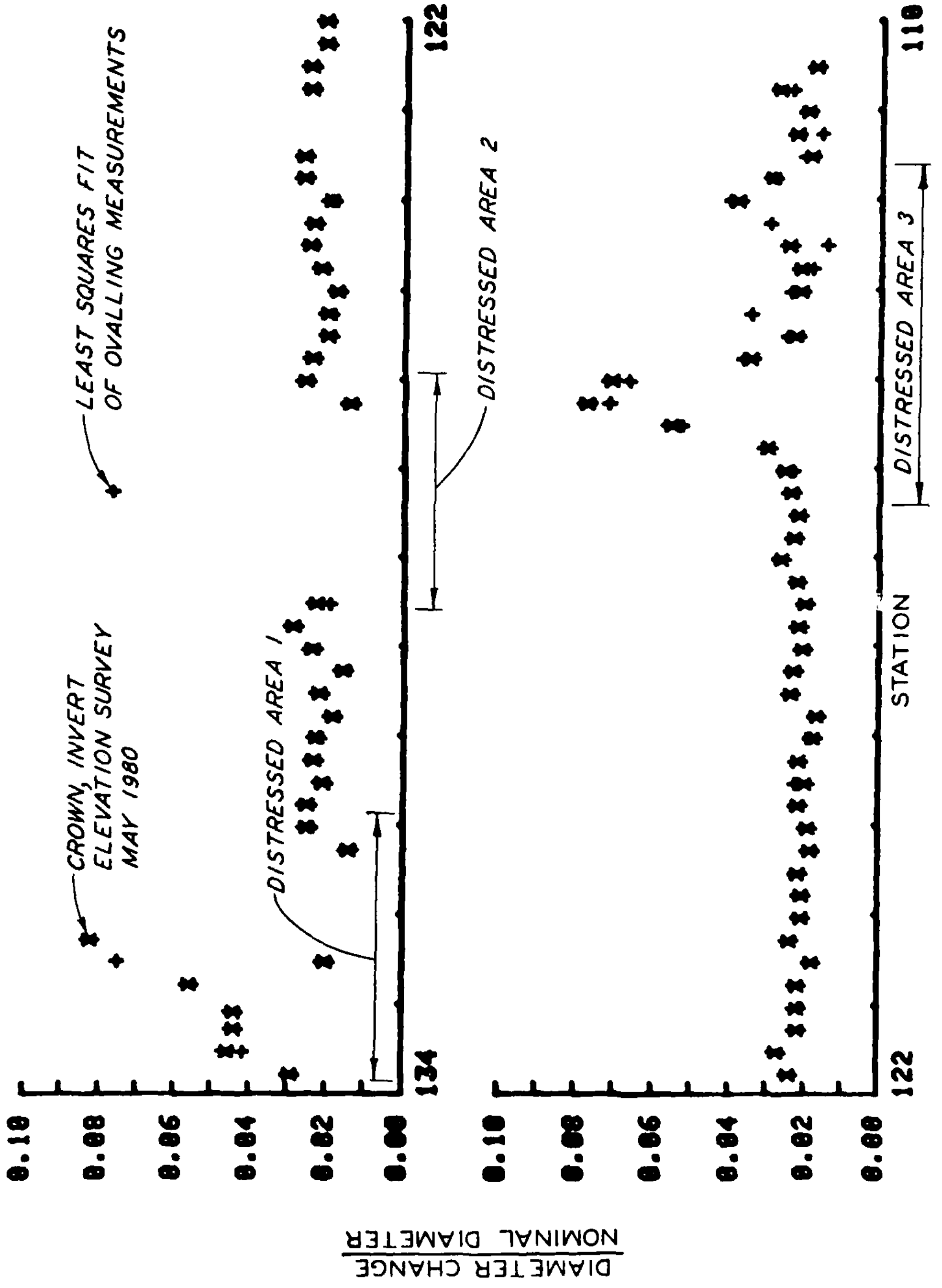


Figure 60. Ovalling profile

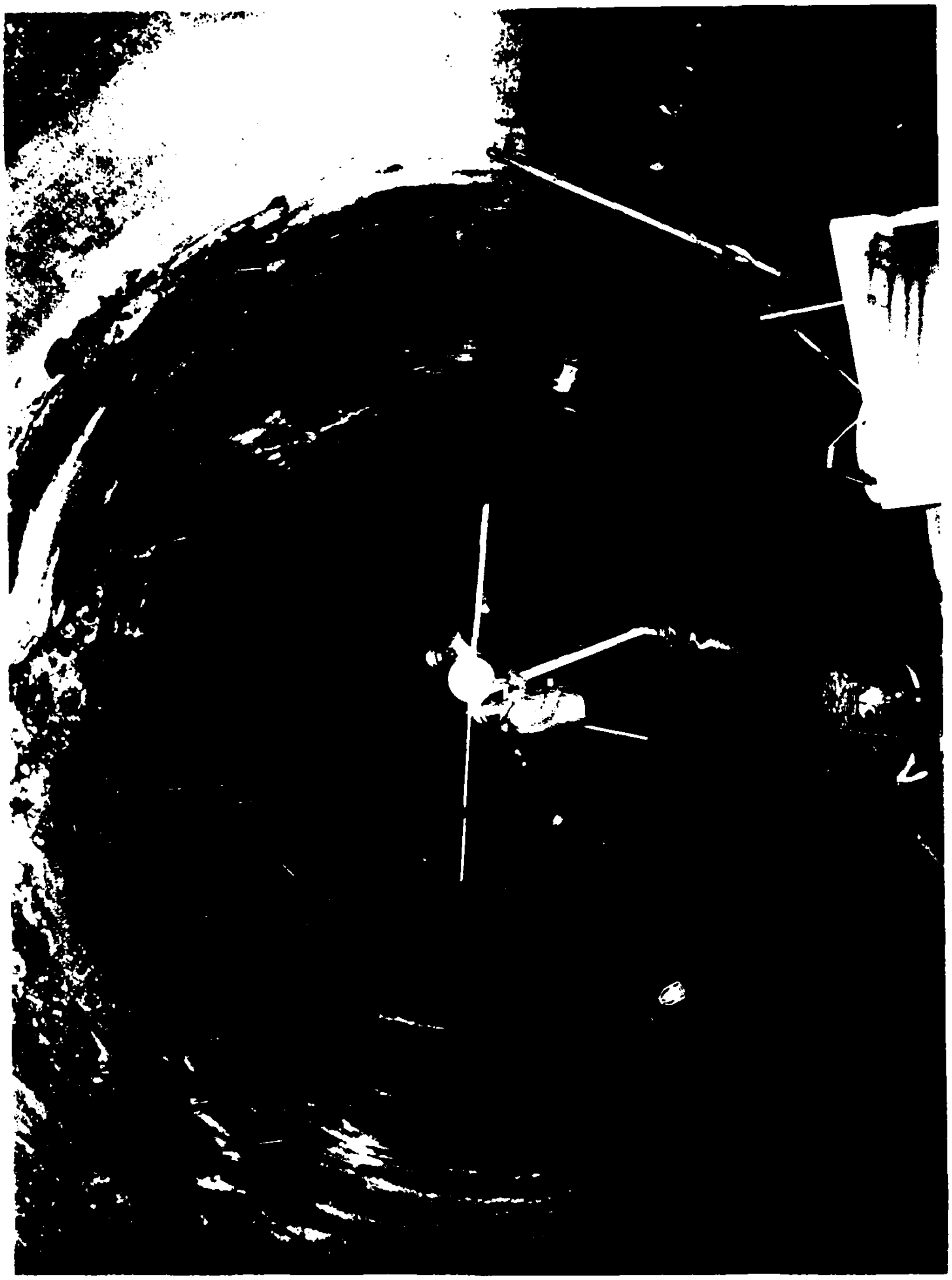
attributable to inner form out of roundness and deformation during construction of the tunnel.

84. This description of the structure's deformation was supplemented by measurements of its ovalled shape in the planes perpendicular to its longitudinal axis at 29 selected locations along its length. These measurements were made in May 1980 by the U. S. Army Engineer District, Detroit. The measurements were obtained by fixing the pivot point of a custom fabricated profile meter at the approximate center of the tunnel as shown in Figure 61. Then the distance from this fixed pivot point to the inner surface of the tunnel along 16 equally spaced rays was recorded.

85. The 29 figures in Appendix F show the results of these measurements. In each figure the solid curve represents the nominal 12-ft-9-in. inner surface of the tunnel. The asterisks depict the actual shape of the cross section measured with the custom fabricated profile meter. The dotted curve is a theoretical periodic deflected shape selected to have the minimum sum of the squared distances from these measurements. The value given for w/r is equal to the maximum diametrical change of this theoretical deflected shape as a fraction of the nominal diameter. It is observed that the springline diameter was stretched to a value greater than the nominal diameter along the entire bypassed section. The crown-invert diameter changes obtained from the ovalling measurements are superimposed on the results of the crown-invert elevation survey in Figure 60. It is seen that the two independently acquired data bases are reasonably consistent with one another.

Design loading

86. In response to a request for design calculations of the failure sewer, the data sheet shown in Figure 62 was the information furnished by the Detroit Water and Sewage Department pertaining to the concrete tunnel walls. In these computations, the loading on the sewer was assumed to be entirely resisted by the concrete wall of the tunnel. This loading was a uniform pressure equal in magnitude to the weight per unit area of the soil between the springline and the



DETROIT WATER SERVICE

SUBJECT OAKLAND-MACOMB INTERCEPTOR SYSTEM

COMPRESSIVE STRESS ON CONCRETE TUNNEL WALLS (SECONDARY LINING)

BY | CHECKED BY | SHEET NO. 1 OF 1 | DATE

CONTRACT No.	SIZE I.D.	WALL t	COVER d *
PCI-4	17'-6"	20"	40'
PCI 5,6 & 7	12'-9"	16"	100'
PCI-8	9'-6"	12"	60'
PCI-9	8'-9"	12"	36'
PCI-12	11'-0"	14"	58'
	10'-6"	14"	61'

* SURFACE TO SPRINGLINE

$$f_c = wd\left(\frac{I.D. + 2t}{2}\right) \div 12t$$

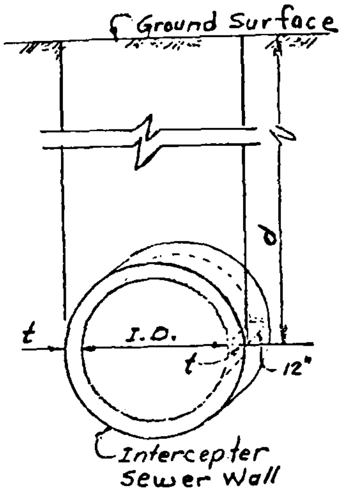
f_c = conc. compr. stress

w = unit wt. of soil = 120 p.c.f.

d = depth of cover (ft.)

I.D. = inside dia. of sewer

t = wall thickness (in.)



Note:
Assumed full earth load without considering arching action nor shearing resistance of soil.

PCI-4

$$f_c = 120 \times 40\left(\frac{17.5 + 2 \times 1.67}{2}\right) \div 12 \times 20 = 208 \text{ psi}$$

PCI-5, 6 & 7

$$f_c = 120 \times 100\left(\frac{12.75 + 2 \times 1.33}{2}\right) \div 12 \times 16 = 482 \text{ psi}$$

PCI-8

$$f_c = 120 \times 60\left(\frac{9.5 + 2 \times 1.0}{2}\right) \div 12 \times 12 = 287 \text{ psi}$$

PCI-9

$$f_c = 120 \times 36\left(\frac{8.75 + 2 \times 1.0}{2}\right) \div 12 \times 12 = 161 \text{ psi}$$

PCI-12

$$f_c = 120 \times 58\left(\frac{11.0 + 2 \times 1.17}{2}\right) \div 12 \times 14 = 275 \text{ psi } (11'\text{-}0")$$

$$f_c = 120 \times 61\left(\frac{10.5 + 2 \times 1.17}{2}\right) \div 12 \times 14 = 280 \text{ psi } (10'\text{-}6")$$

Figure 62. Original design calculations

ground surface. An average compressive stress of 482 psi in the trial 16-in.-thick concrete wall consistent with this uniformly distributed pressure was then computed. Under the presumed loading q, the axial thrust N resulting from this compressive stress would uniformly compress the tunnel's circular cross section as shown in Figure 63. This assumed behavior differs from the pattern of crown-invert cracking and springline spalling observed in the distressed areas of the tunnel.

<u>Resistance specified</u>

87. The contract specifications prescirbed a tunnel wall thickness of 16 in., a minimum cement content, a maximum water content ratio, and an approximate ratio of fine to coarse aggregate for the concrete mixture to be used in the tunnel walls, but did not explicitly require the mixture to meet any strength standard. We have, however, been told by the Detroit Water and Sewer District that the compressive strength of the concrete from similar projects using this particular mixture is 3500 psi. This strength provides a comfortable margin of safety against the uniform loading assumed in the design.

88. As the distressed areas of the tunnel may have experienced a loading condition different from the uniform condition assumed in its design, it is appropriate to consider the inherent resistance of the tunnel walls against nonuniform loadings q, as shown in Figure 64. In order to satisfy the conditions of static equilibrium for such loadings, the thrust N must now vary with position in the tunnel wall. Further, transverse resultant shear forces V and resultant bending moments M must be developed which vary with position in the tunnel wall. Bending moments acting in the direction of those in Figure 64 would tend to flex the tunnel wall inward as indicated and cause cracking on the inner surface of the structure as observed near the crown and invert of the distressed areas. On the other hand, bending moments acting opposite to the direction of those in Figure 64 would tend to flex the tunnel wall outward and cause crushing and eventually spalling on the inner surface of the structure as observed near the springline of the distressed areas.

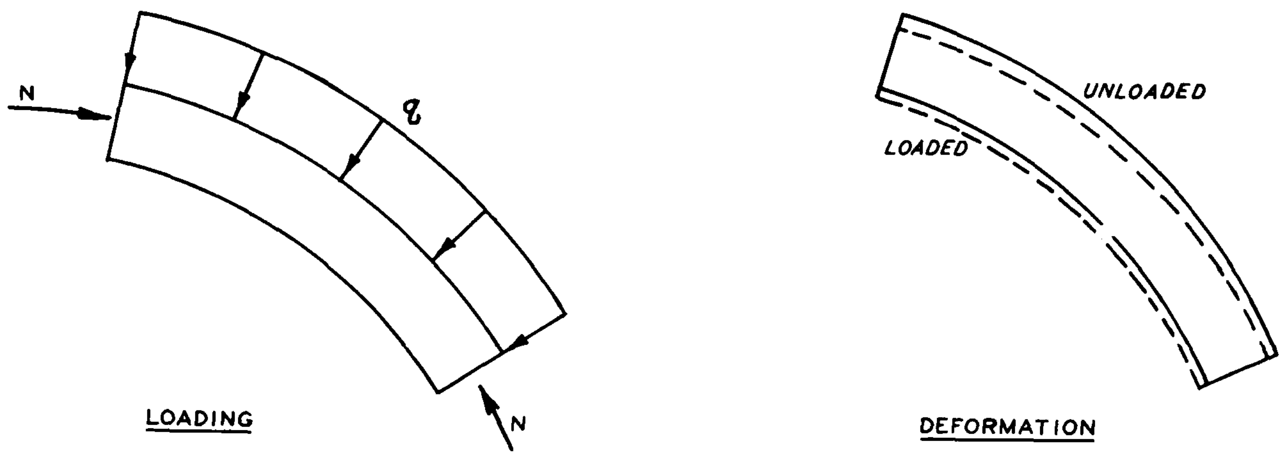
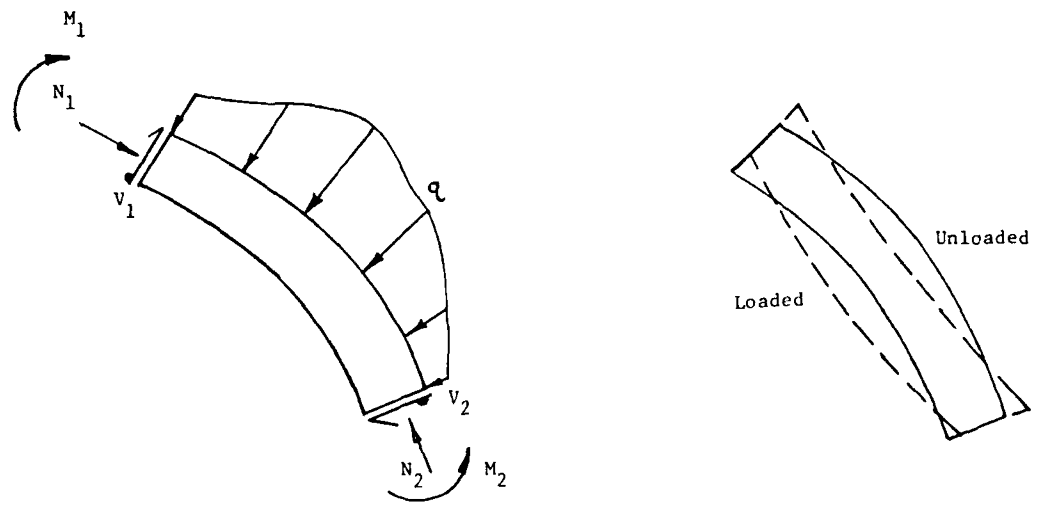


Figure 63. Behavior of circular tunnel wall under uniform external pressure



Loading

Deformation

Figure 64. Behavior of circular tunnel wall under nonuniform external pressure

89. When loaded in such a nonuniform fashion, the resistance of a plain concrete tunnel wall depends upon its tensile strength as well as its compressive strength and thickness. The specific combinations of moment M and thrust N which can be resisted without concrete cracking or crushing can be computed from the values of tensile strength, compressive strength, and wall thickness using the techniques discussed by Pfrang, et al (1964). These techniques involve first assuming a set of linear strain distributions across the wall corresponding to a cracking or a crushing failure; then, for each strain distribution, the distribution of stress is computed from the stress-strain relation for concrete which is approximated by a parabola in compression and a straight line in tension. Finally, the axial thrust N and bending moment M which are statically equivalent to this stress distribution are calculated. A TEKTRONIX 4050 series BASIC mini-computer subroutine program to perform these computations is listed in Appendix G. For the concrete mixture specified, it was estimated that the tensile strength would be about 8 percent of the compressive strength or 280 psi (Grieb and Werner, 1962). From these strength and thickness values, the resistance of the structure specified by the contract was then computed to be the dotted line of Figure 65. Combinations of bending moment M and axial thrust N caused by the loading which fall between this line and the coordinate axes can be resisted by the tunnel wall, while combinations of moment and thrust in excess of this resistance line cause the failure of the tunnel wall. This resistance line indicates that the specified structure offers some resistance against nonuniform loading although this was not explicitly considered in the design calculations furnished us. In the absence of any axial thrust, this resistance is small due to the low strength of concrete in tension. However as the axial thrust increases up to the point of inducing crushing of the concrete in compression, the resistance to bending moments induced by nonuniform loading increases.

Resistance constructed

90. As discussed in the previous section, the resistance against

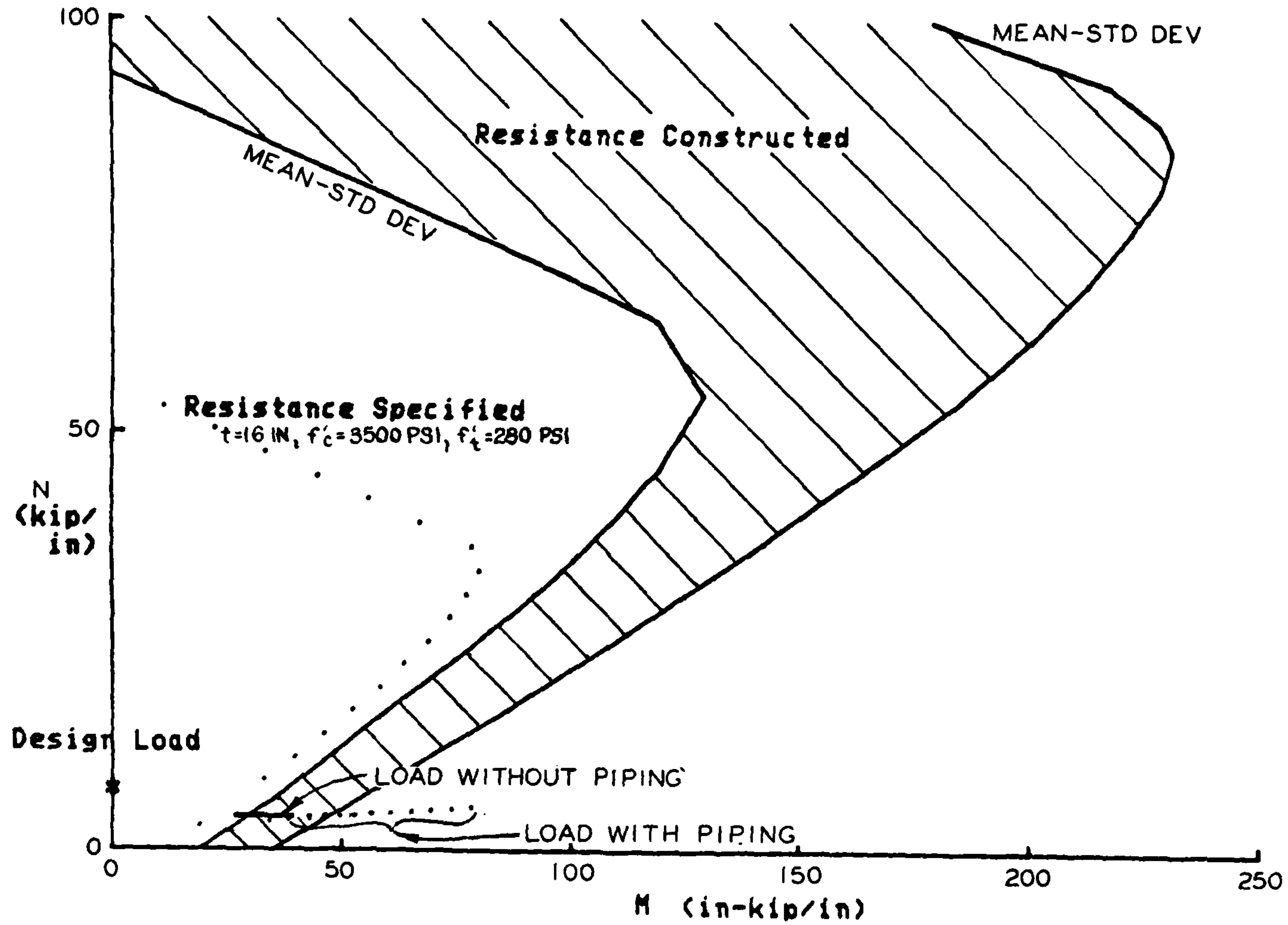


Figure 65. Structural reliability of concrete lining under external pressure

external loading offered by a plain concrete section is determined by the concrete strength and the section thickness. Figure 66 is a histogram which shows the distribution of concrete compressive strength in the tunnel. This figure was drawn from the results of the laboratory tests which were discussed in the concrete material properties portion of this report. Figure 67 is a histogram of section wall thickness which was obtained from the core measurements made during the geotechnical explorations through the tunnel wall as well as these concrete material investigations. It is clear that the resistance offered by the structure as constructed varies throughout the tunnel in accordance with the distributions of these histograms and the dependence of resistance on strength and thickness.

91. In order to estimate this distribution of resistance in the tunnel as built, a probabilistic technique called Monte Carlo simulation (Benjamin and Cornell, 1970) was employed as follows. First, the concrete compressive strength was modeled by a computer-generated value of a random variable whose theoretical distribution roughly approximated that of Figure 66 and was independent of location in the tunnel. Specifically the mean and standard deviation of the theoretical distribution, which are measures of its central value and dispersion, respectively, were set equal to those of the sample shown in Figure 66. Similarly, a computer-generated value for section thickness was obtained from a theoretical distribution having the same mean and standard deviation as the sample in Figure 67. Next, a tensile strength was generated from a theoretical distribution whose mean was 8 percent of the compressive strength which has been previously explained to be the estimated central value for this mix design. The standard deviation of this distribution was assumed to be 13 percent of this mean value as this is the ratio of standard deviation to mean for the sample of concrete compressive strength in Figure 66. From these simulated values of strength and thickness, a bending moment and axial thrust resistance curve was then drawn just as the dotted line was drawn for the contract specified values of these variables in Figure 65. This simulation procedure was replicated by a computer

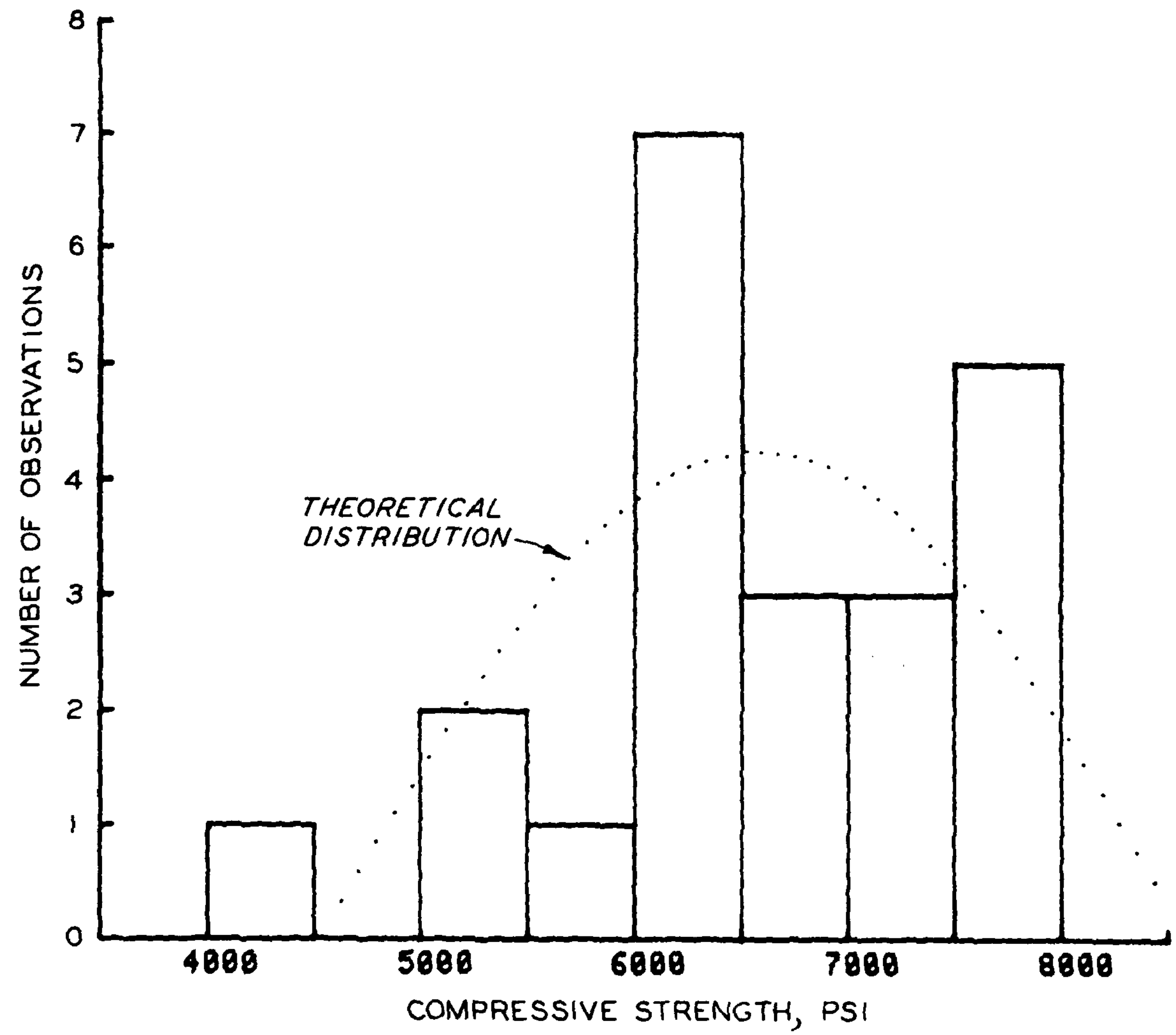


Figure 66. Histogram - compressive strength of drilled concrete cores (psi)

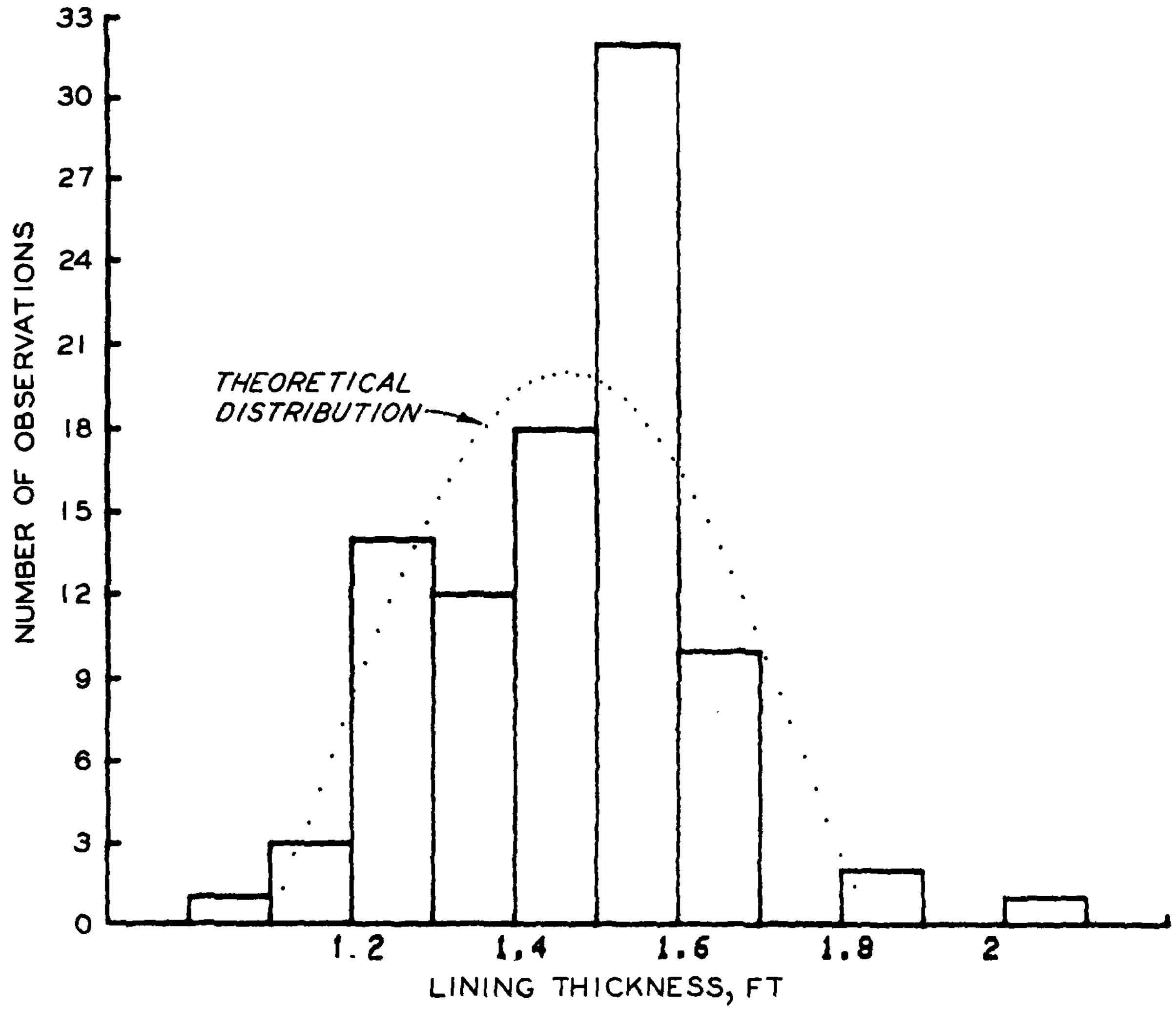


Figure 67. Histogram - thickness of concrete liner (ft)

100 times resulting in 100 such resistance curves. Finally, the mean and standard deviation of these 100 curves were computed. The two solid curves in Figure 65 represent the mean plus one standard deviation and the mean minus one standard deviation resistances obtained by this procedure.

92. The hashed region between these solid curves is expected to contain the resistance constructed in the majority of the sewer. However, the structure's resistance is probably greater or less than these curves at some locations. One finds from this figure that the as-built resistance generally exceeded the resistance specified by the contract.

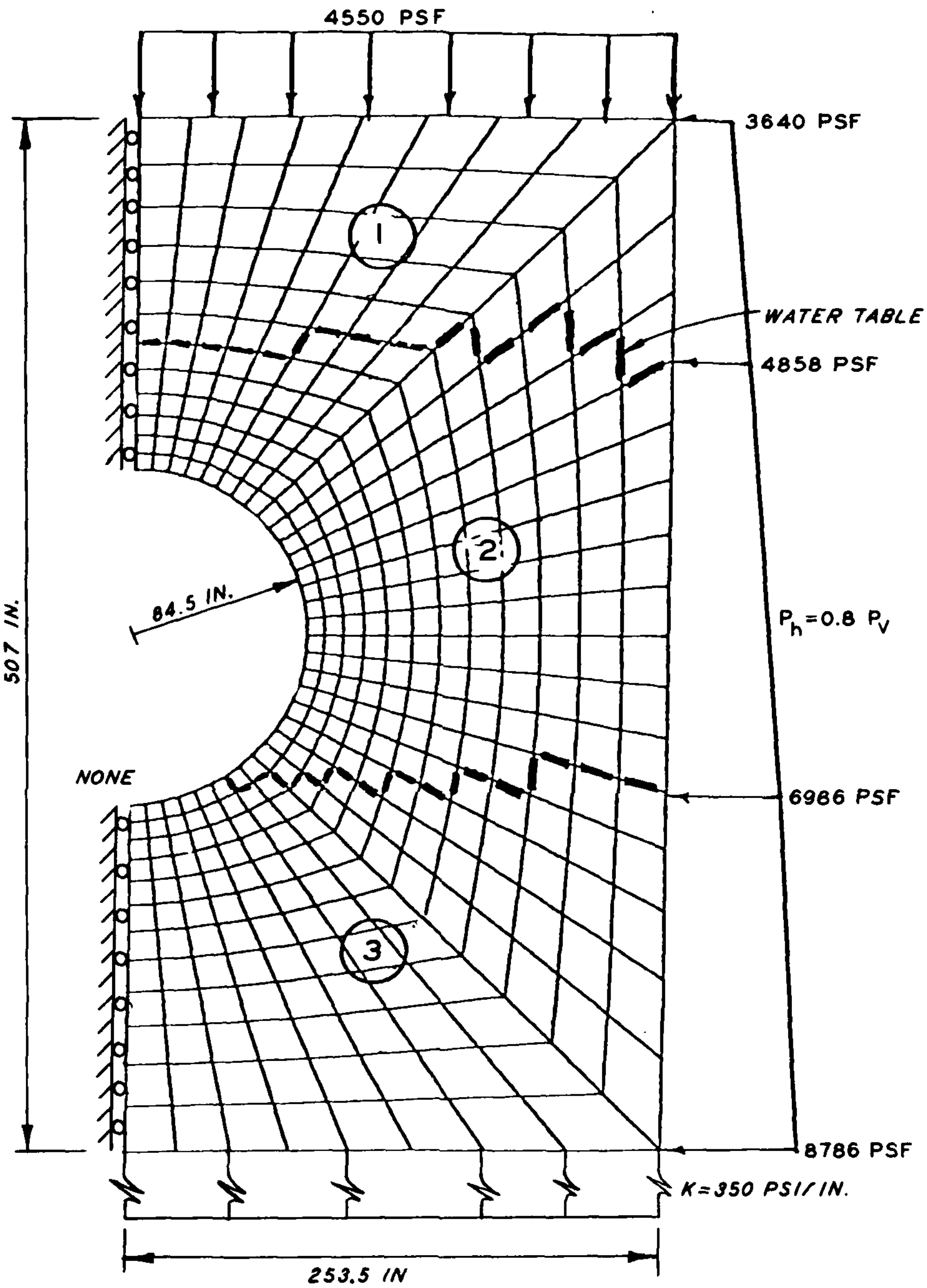
Loadings imposed

93. As discussed by Peck (1969), the loads imposed on any tunnel are the result of a complex interaction mechanism which takes place between the lining structure and the medium surrounding it. At one hypothetical extreme, if the lining is perfectly flexible compared to the medium, the lining will distort until a uniform pressure exists between the lining and the medium. This lining will satisfactorily support the surrounding soil if the associated axial thrust does not induce crushing or buckling. At the other theoretical extreme, if the lining is absolutely rigid relative to the soil, the nonuniform pressure distribution which would exist in the undisturbed medium is imposed on the lining. To perform acceptably, this lining must be capable of resisting the internal bending moments induced by this nonuniform loading. In the present case, the primary rib and lagging lining preserved the integrity of the excavation during the construction of the secondary concrete lining. Any post-construction change in loading, such as that caused by the removal of dewatering wells or by the piping of surrounding material, must be shared by the two linings in accordance with the rigidities. Studies such as that by Meyer and Flathau (1967) have shown that when a partial failure of the buried structural system occurs, these interaction mechanisms can sometimes change drastically.

94. With these complications in mind, stress analyses were

conducted of the combined structure-soil system to bracket the loadings imposed on the concrete lining by the various geotechnical conditions discussed in an earlier portion of this report. Since the concrete lining was constructed in intimate contact with the primary rib and lagging lining and since the concrete lining has approximately 100 times the flexural rigidity of the primary ribs, it was assumed that all of the external loading following construction was carried by the concrete lining. Accordingly, these analyses are useful in discussing the initiation of failure in the concrete lining and do not necessarily reflect in detail the complex interaction among concrete lining, primary lining, and surrounding medium to the point of severe structural collapse which now exists in some areas in the sewer.

95. The interaction between the concrete lining and the surrounding soil was modeled by loading the soil at a distance of one tunnel diameter from the structure with the loading which would exist at this location if there were no structure present as shown in Figure 68. In these stress analyses both the soil and structure were assumed to behave with linear elastic material properties consistent with previously discussed test data and with compatible displacements at the soil structure interface. The effects of the stratification and the piping discussed in the geotechnical portion of this report were modeled through a nonhomogeneous distribution of soil material properties. To simplify the analyses, the loading conditions considered were assumed to be symmetric about the tunnel's vertical axis as shown. Along the horizontal boundary located one tunnel diameter above the crown, a uniform vertical pressure was applied corresponding to the weight of the overburden material as previously computed in Figure 40. On the vertical boundary one tunnel diameter from the springline, a horizontal pressure was applied corresponding to K_p as indicated in Figure 41. The horizontal boundary one diameter below the invert was assumed to rest on a linear elastic foundation whose modulus $k = 300$ psi/in. was selected to be representative of the soil material encountered below this depth. In addition to these boundary loadings, the weight distribution of the lining and soil material within these



SOIL PROPERTIES

MATERIAL	E, PSI	ν	γ, PCF
1. SILT & CLAY	3000	0.44	138
2. SILT & CLAY	3000	0.44	140
3. SAND	6000	0.44	125

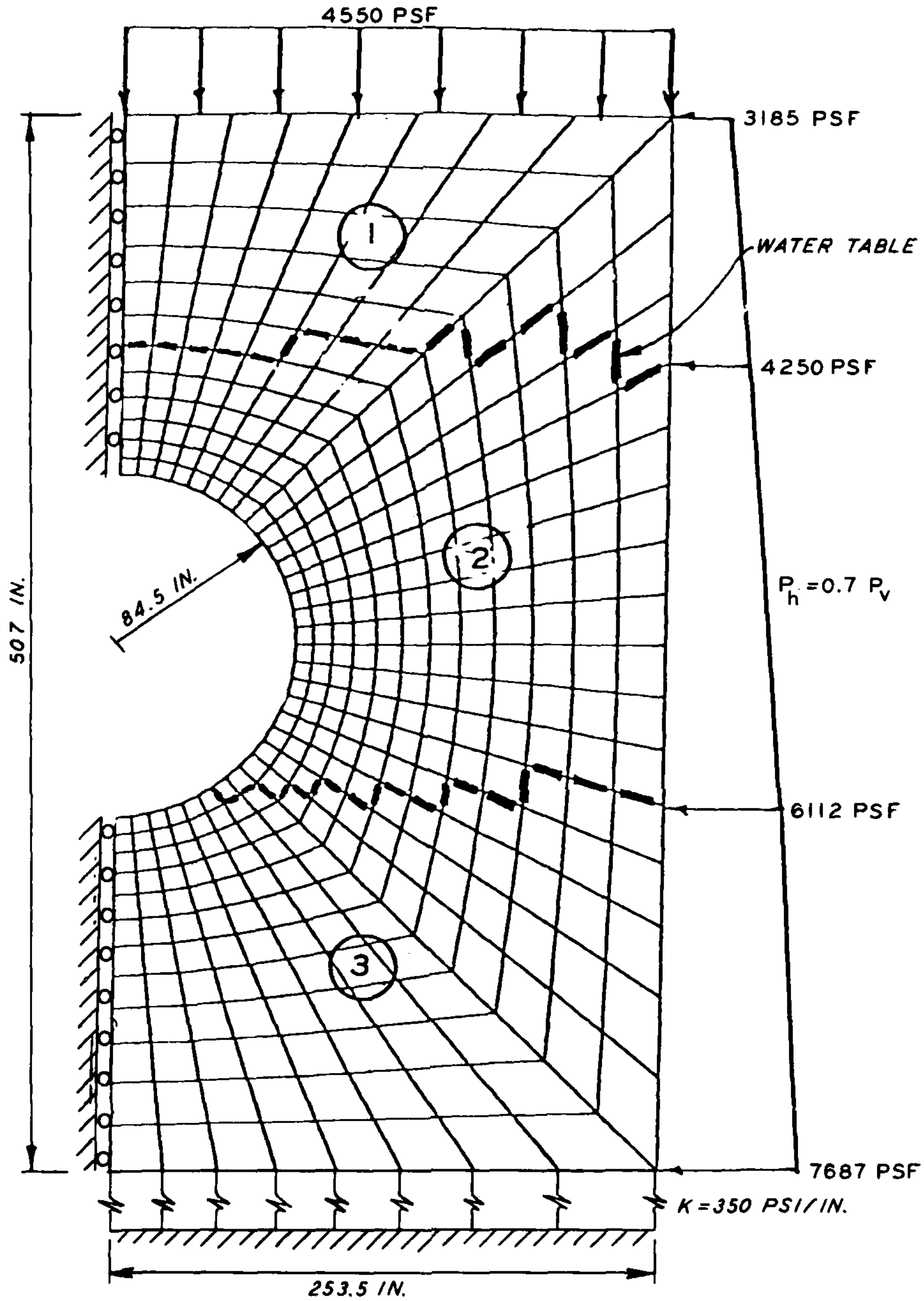
Figure 68. Postconstruction loading conditions; no piping, K = 0.8

boundaries was also included in the analyses.

96. These stress analyses were conducted using a numerical technique called the finite element method (Desai and Abel, 1972). In this method, constant material properties and a particular form of the distribution of stress is assumed within each quadrilateral element of the grid in Figure 68, but the magnitude of these stresses is determined by the requirement for static equilibrium with the externally applied loads. The analyses were performed on the WES digital computer using the computer code SAPV (Bathe, et al., 1973). The soil materials were modeled by isoparametric, plane strain, quadrilateral elements with incompatible bending modes of deformation suppressed. The concrete lining was modeled by beam elements which considered flexural, shear, and axial flexibilities, and which were located at the linings nominal central 84.5-in. radius. In all the analyses, the stiffness properties of these beam elements were computed for a nominal thickness of 16 in., a modulus of elasticity of $E = 6 \times 10^6$ psi and a Poisson's ratio of $\nu = 0.20$.

97. The first two analyses were conducted to bracket the loading imposed on the structure if no surrounding material piped into the sewer. The soil stratification and loading in these analyses are shown in Figures 68 and 69. The stratigraphy shown is a simple idealization of the complex condition which has been discussed earlier in this report. The loadings assumed in these two cases bracket those associated with the range of lateral earth pressure coefficient $0.7 < K < 0.8$ previously determined for the surrounding material.

98. The results of these analyses are summarized in Figures 70 through 73 as a function of location from the invert at $\theta = 0$ to the springline at $\theta = \pi/2$ to the crown at $\theta = \pi$. The pressure q exerted on the liner by the surrounding soil is seen to be nonuniform due to the variation of load with depth and the stratification of the soil. The shearing stress τ exerted at and parallel to the structure soil interface is small relative to these normal pressures and could probably be transmitted without slip along this surface as assumed in the analyses. The bending moments M induced in the tunnel wall by



SOIL PROPERTIES

MATERIAL	E, PSI	ν	γ, PCF
1. SILT & CLAY	3000	0.41	138
2. SILT & CLAY	3000	0.41	140
3. SAND	6000	0.41	125

Figure 69. Postconstruction loading conditions; no piping, K = 0.7

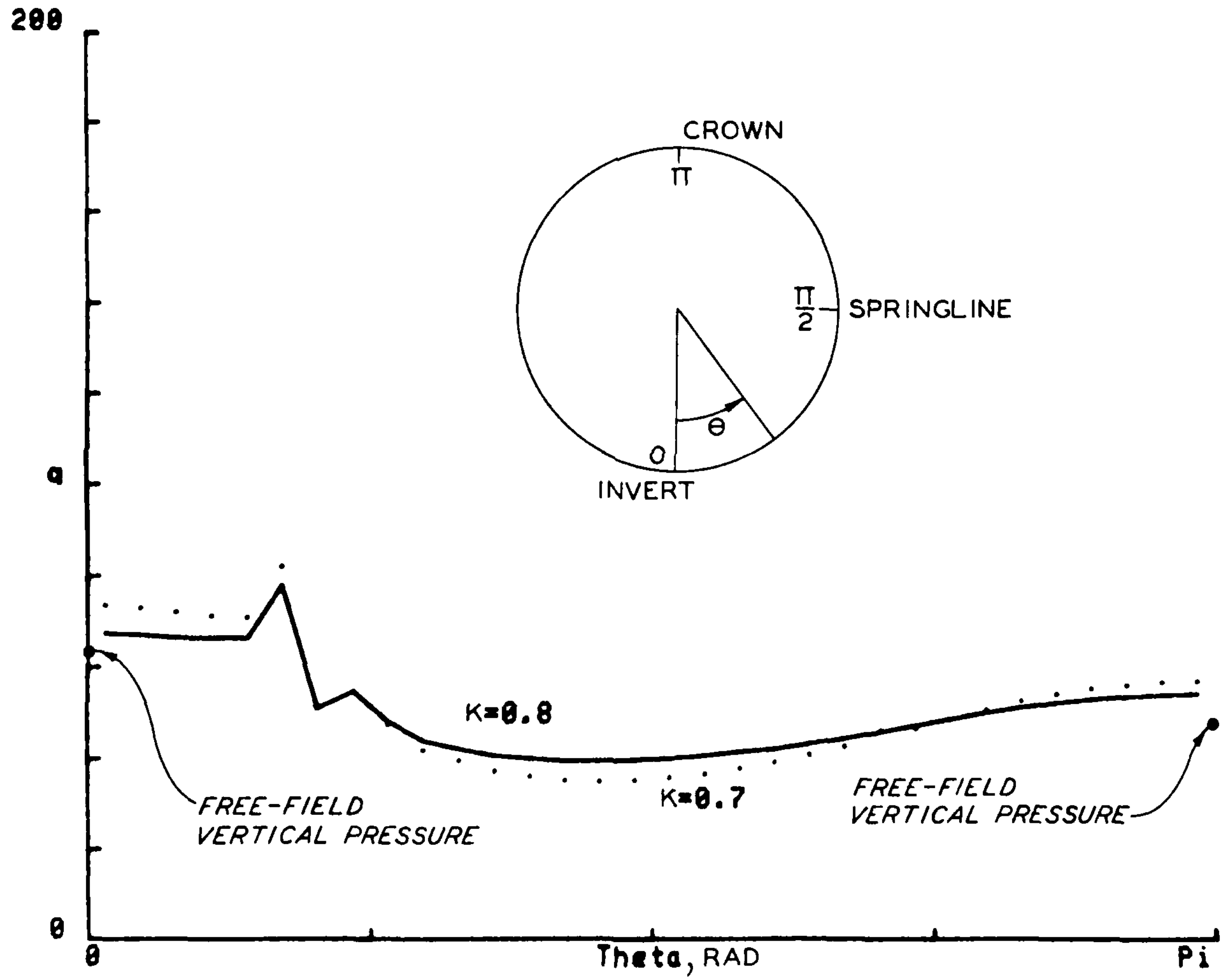


Figure 70. Pressure distribution on tunnel liner; no piping

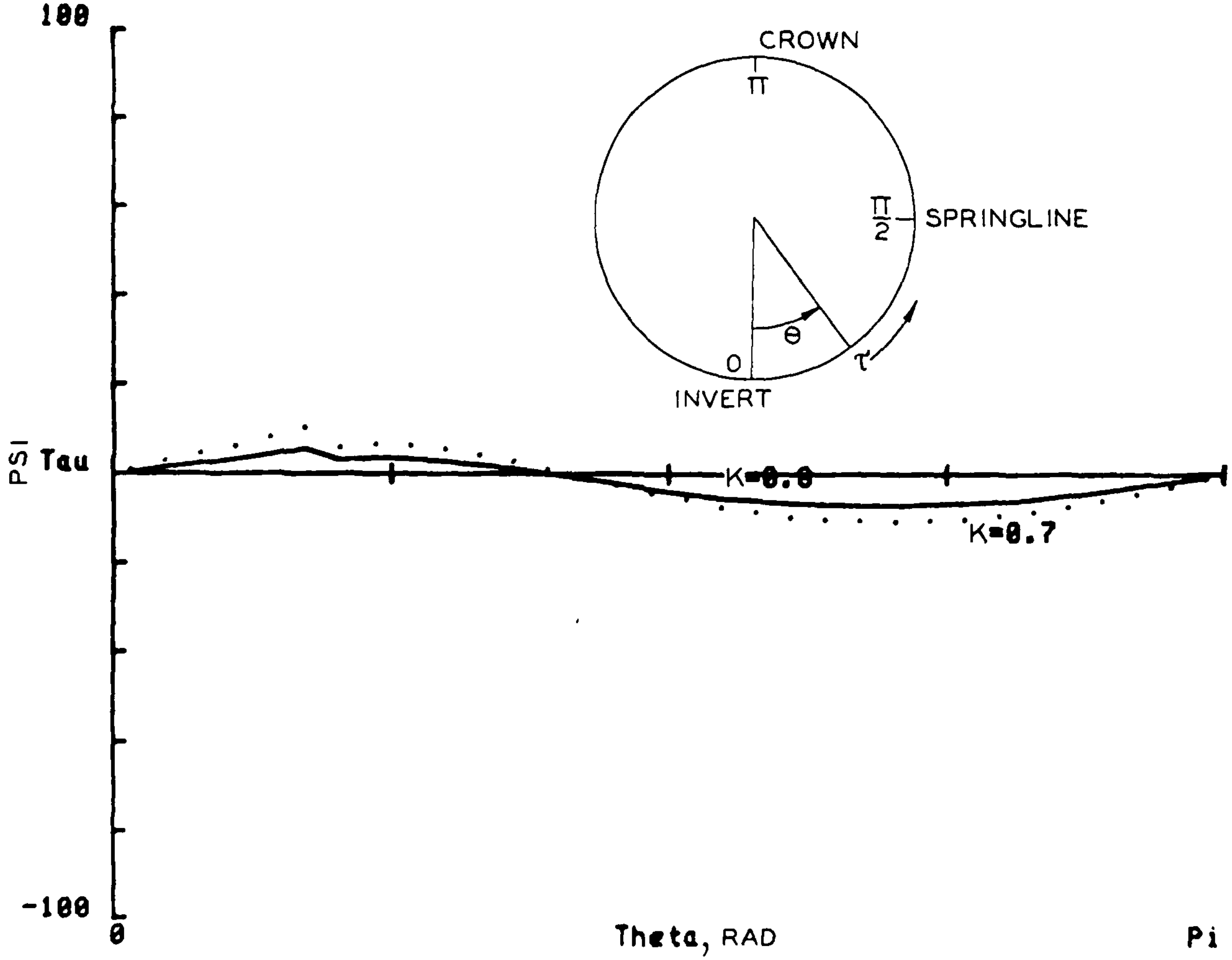


Figure 71. Interface shear distribution; no piping

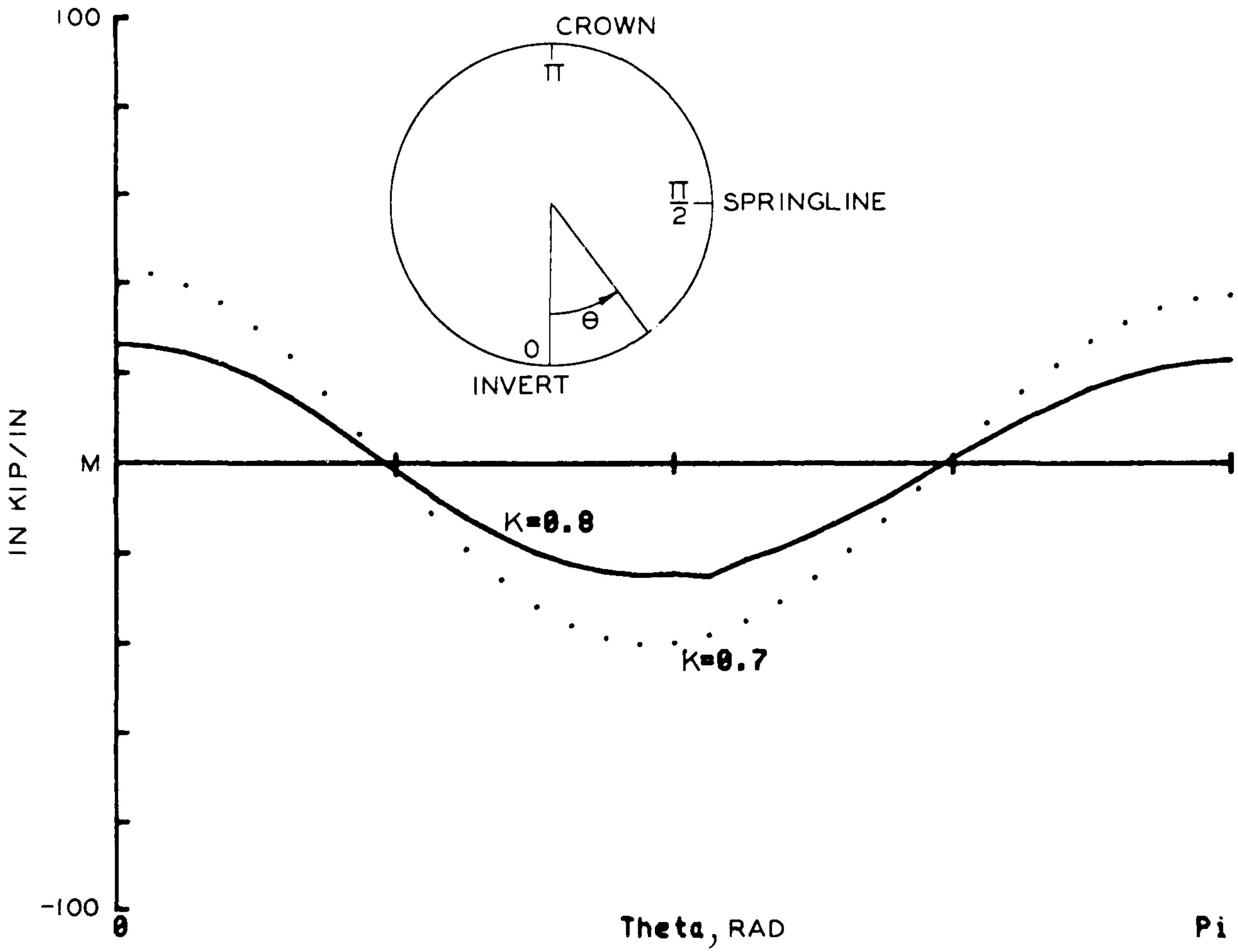


Figure 72. Internal bending moment distribution; no piping

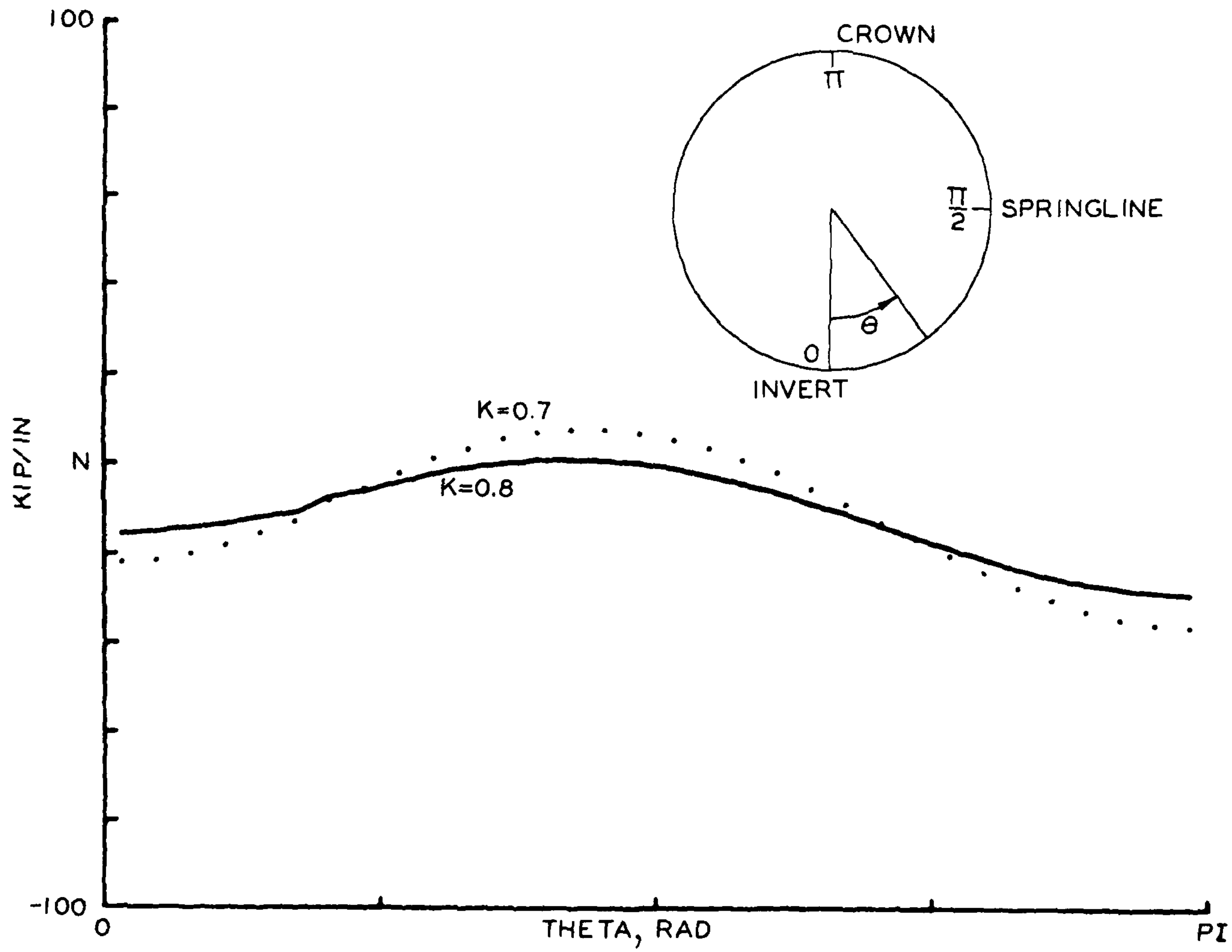
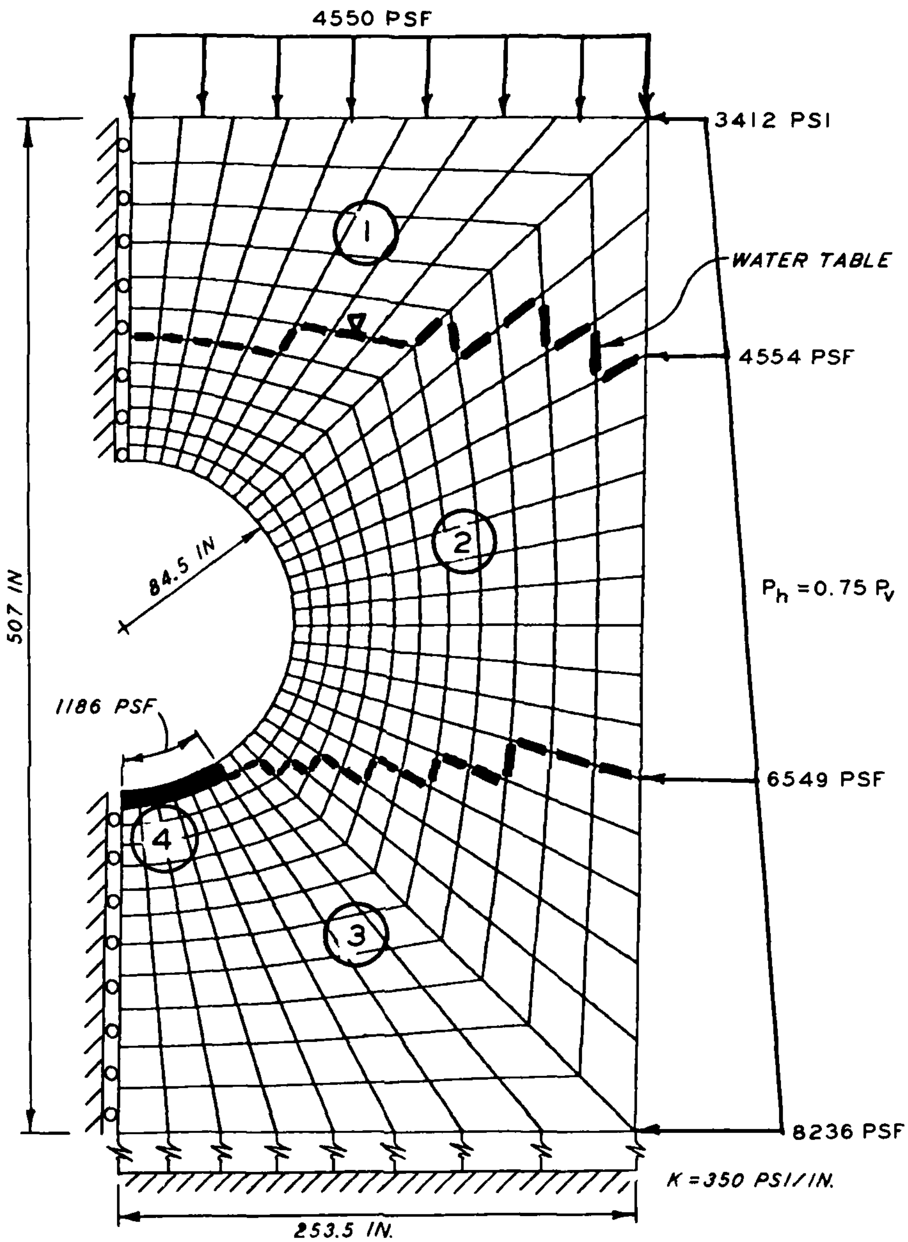


Figure 73. Axial thrust distribution; no piping

these loadings are noted to stretch the inner surface of the liner near the invert and crown and to compress this surface near the springline. The corresponding axial thrusts N are observed to vary as a function of position in the lining to maintain equilibrium with the nonuniform loading.

99. In both of these loadings the most severe combinations of moment and thrust occur at the invert, crown, and springline. The severity of this combination increases as the lateral pressure coefficient K decreases. The range of these ideal invert loading combinations has been superimposed on the previously drawn resistance curves of Figure 65. It is not believed that this range of loadings would exceed the capacity of the constructed tunnel system. Figure 70 does admit that if the concrete lining in the weaker areas of the sewer tunnel was surrounded by piped material with a low value for the lateral earth pressure coefficient, some minor cracking could have initiated in these areas. Some of the loading would then be transferred from the concrete lining to the rib and lagging lining. A comparison of Figures 40 and 41 indicates that this primary lining was subjected to a more nonuniform loading condition during the tunnel's construction. Accordingly, the primary lining and surrounding competent soil material could be expected to intersect and successfully carry this post-construction loading.

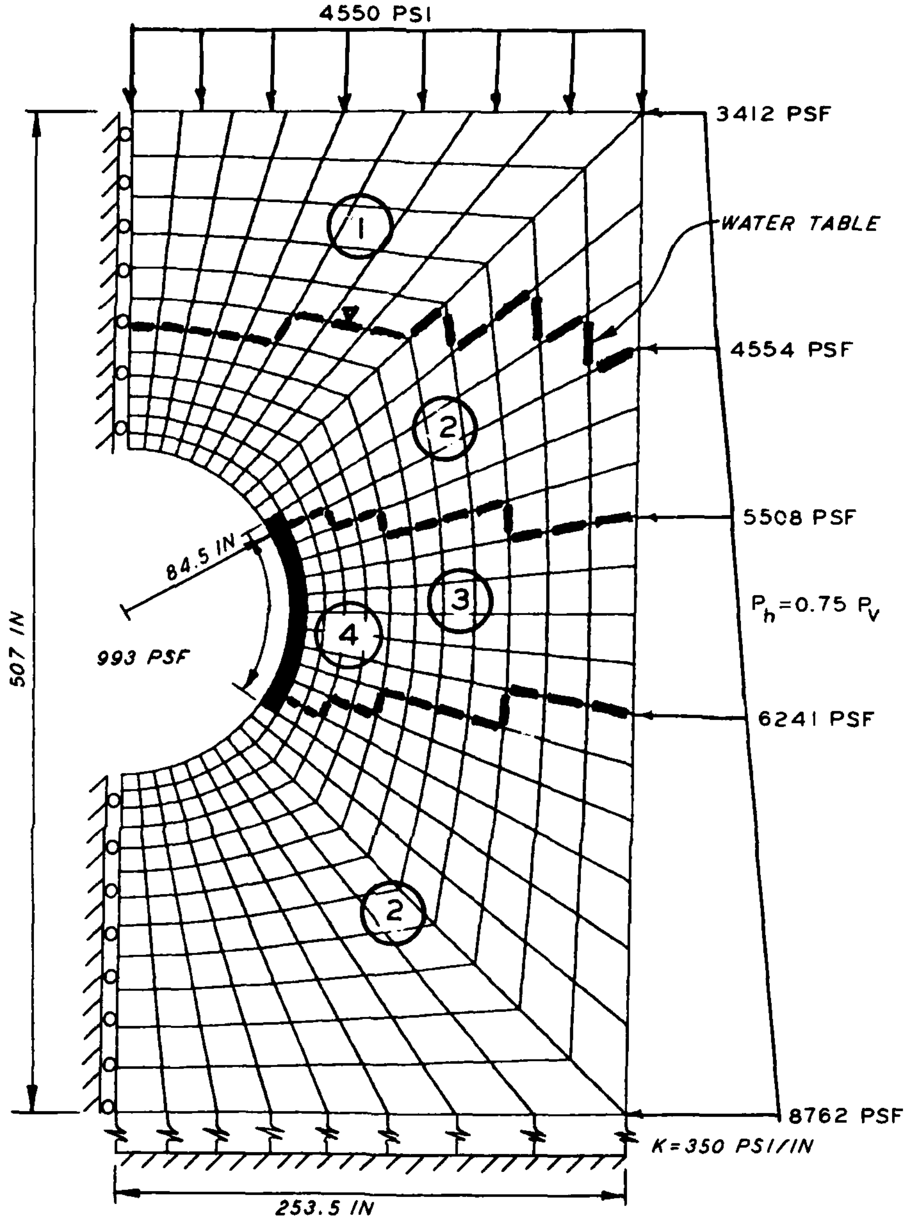
100. Two other analyses were then conducted to bracket the loading imposed on the structure if some of the surrounding material piped into the sewer. The stratigraphy and loading assumed in these analyses are shown in Figures 74 and 75. The soil stratification shown represent two simple idealizations which bracket the complex layering of pipable material which have been previously shown to exist around some areas in the tunnel. The boundary loadings assumed in these two cases are consistent with the estimated central value K = 0.75 for lateral earth pressure coefficient. In Figure 74 the effect of piping-susceptible material near the invert is modeled by a reduction in this material's stiffness and by a void adjacent to the concrete lining. In Figure 75 the layer of pipable material is moved to the springline



SOIL PROPERTIES

MATERIAL	E, PSI	ν	γ, PCF
1. SILT & CLAY	3000	0.43	138
2. SILT & CLAY	3000	0.43	140
3. SAND	3000	0.43	125
4. VOID	1	0.43	1

Figure 74. Postconstruction loading conditions; invert piping



SOIL PROPERTIES

MATERIAL	E, PSI	ν	γ, PCF
1. SILT & CLAY	3000	0.43	138
2. SILT & CLAY	3000	0.43	140
3. SAND	3000	0.43	125
4. VOID	1	0.43	1

Figure 75. Postconstruction loading conditions; springline piping

and the effect of its piping is similarly simulated at that location. In both cases, the pressure exerted on the lining by the water in these voids is taken to be the product of the density of water and the depth below the water table as indicated.

101. The results of these analyses are summarized in Figures 76 through 79. In both cases the interface pressure q and the shearing stress τ across the structure medium interface are seen to be markedly nonuniform near the voids assumed to be caused by piping. The effect of this loading on bending moment M and axial thrust N is not particularly harmful for the case of a void under the invert. In this case the regions in which the tunnel's inner surface is stretched or compressed change from the unpiped cases, but the severity of the critical moment and thrust combination does not change appreciably from these unpiped cases. On the other hand, if a void occurs near the springline, the severity of the critical moment and thrust combination increases dramatically from the unpiped case. In this case the inner surface of the tunnel near the springline is compressed while the corresponding surfaces near the invert and crown are stretched. This pattern of deformation is similar to the distressed condition existing in the tunnel which has been previously discussed in paragraph 82.

102. These critical moment and thrust loading combinations are connected by a dotted line on the previous plot of resistance curves in Figure 65. This range of loadings suggests that piping of material near the invert would probably not directly cause the collapse of the constructed tunnel system. Such piping could possibly initiate the structural failure of the concrete lining in the weakest areas of the sewer. However if such a failure did begin, it is again believed that the subsequent interaction among the concrete lining, primary lining, and the competent soil material remaining adjacent to the springline could have prevented the severe structural failure of the tunnel which did occur in some areas.

103. In contrast, Figure 65 suggests that the piping of material near the springline would be expected to initiate the structural failure of the concrete lining in even the strongest areas of the

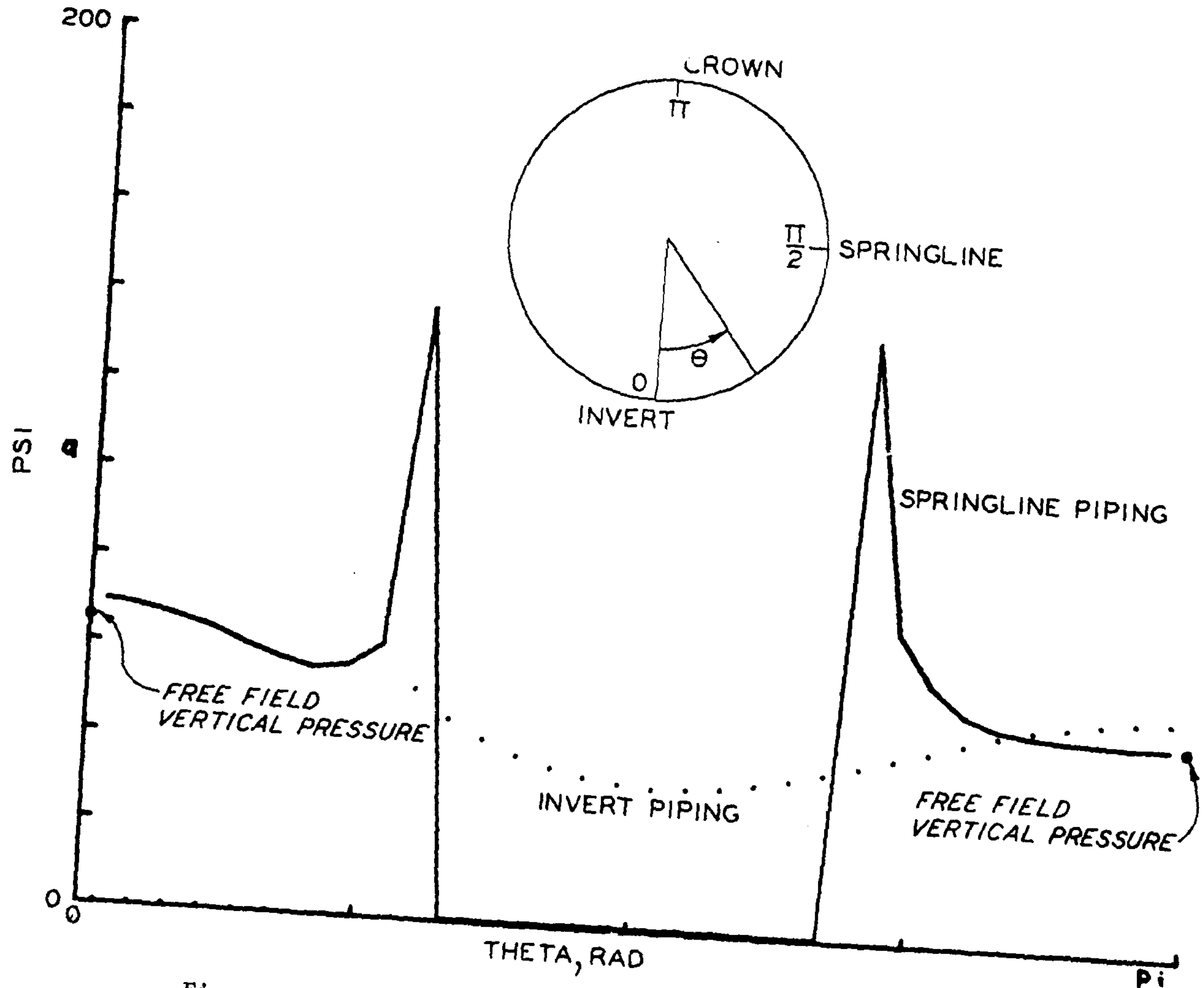


Figure 76. Pressure distribution on tunnel liner; piping

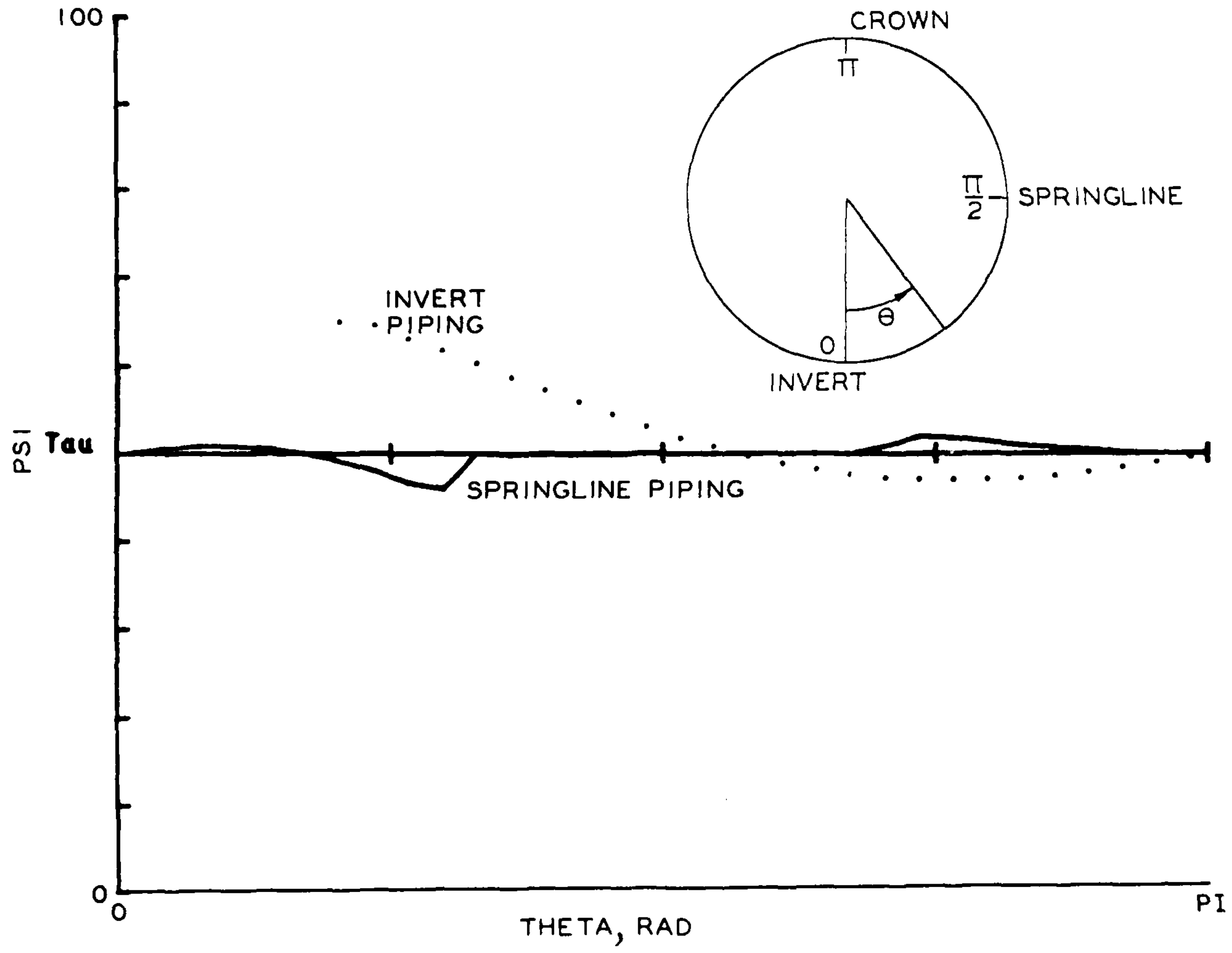


Figure 77. Interface shear distribution; piping

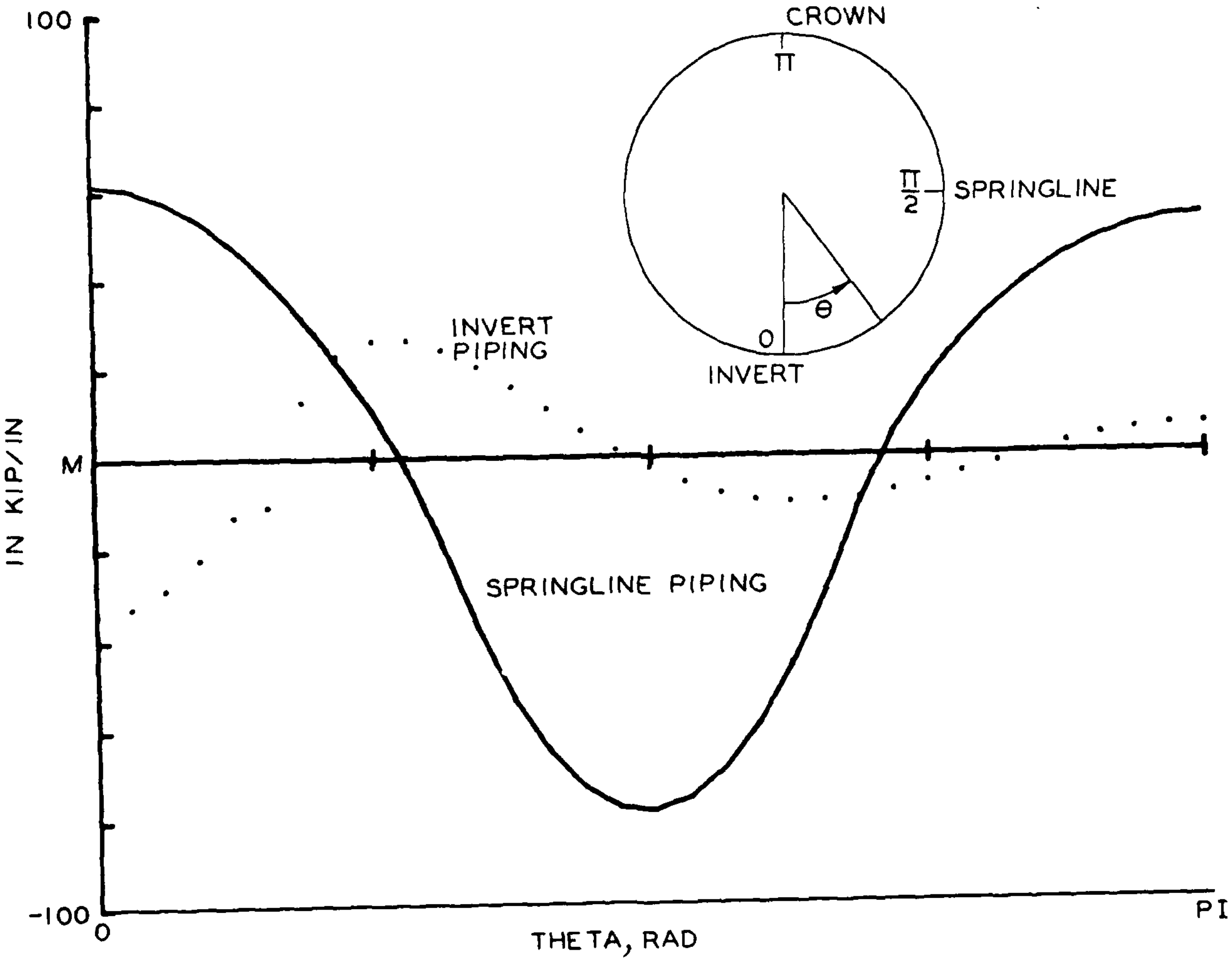


Figure 78. Internal bending moment distribution; piping

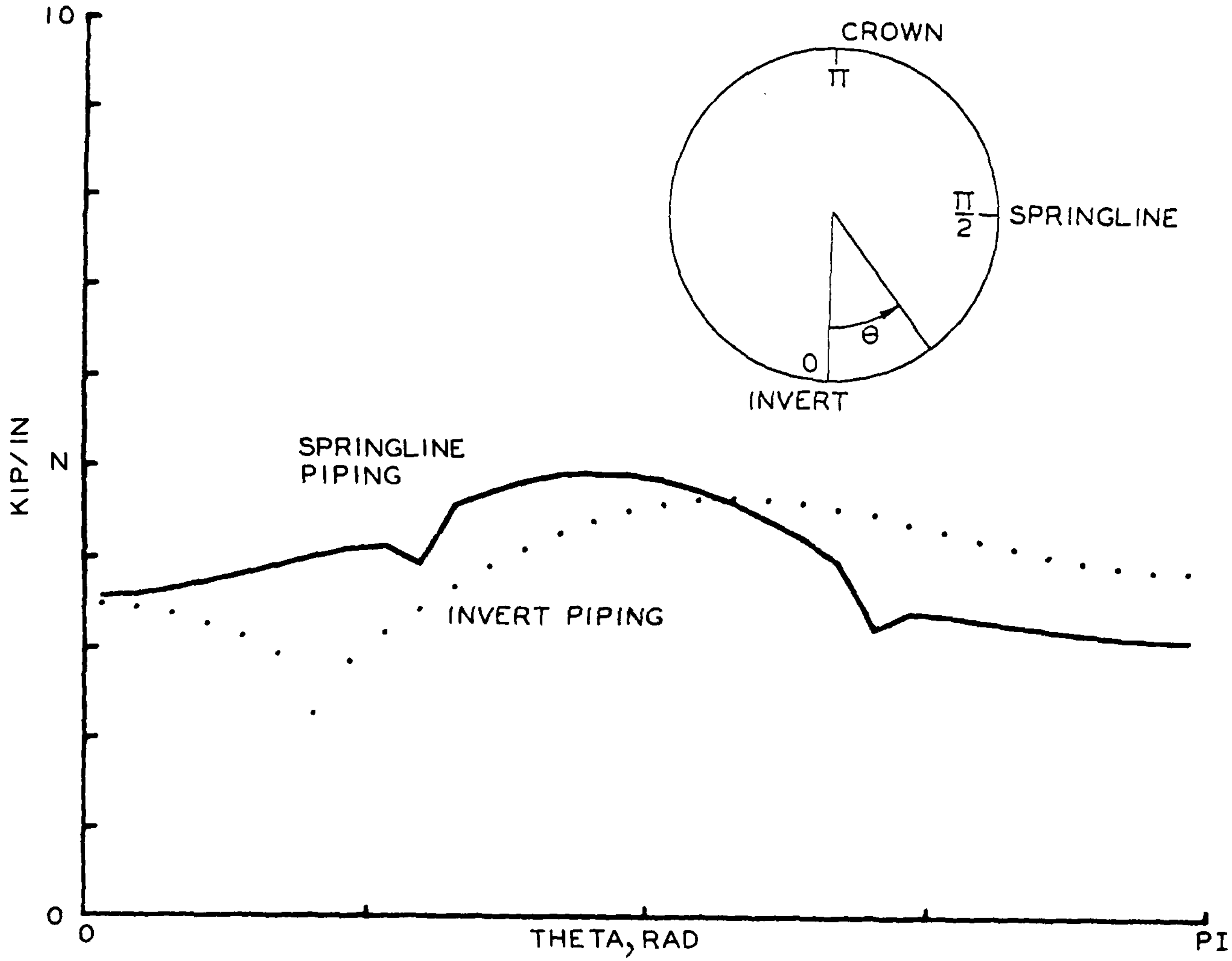


Figure 79. Axial thrust distribution; piping

sewer. If such a failure did initiate, some of the loading would attempt to transfer from the concrete lining to the primary rig and lagging lining. However, lacking any competent soil material adjacent to the springline with which to interact, this flexible primary lining would have little capacity to sustain this loading. Eventually, as severe a collapse of the tunnel as did occur in the distressed areas could be expected.

104. One additional stress analysis was conducted to assess the effect of a continuous void extending longitudinally along the invert due to extensive piping of material. In this analysis the concrete lining was modeled to be an infinitely long flexural beam of rigidity EI as shown in Figure 80. The beam is loaded by a uniform pressure p corresponding to the weight of the overburden material above it and it is supported by an elastic foundation whose modulus k represents the stiffness of the soil material on which it rests. A void of length L exists under the beam to model a complete loss of support due to piping of surrounding material. Using the methods described by Timoshenko (1930), one can derive the expression given in Figure 80 for the bending moment M imposed in the beam at the center of the void. This moment would tend to crack the lining circumferentially rather than longitudinally as would the previously discussed moments induced by nonuniform external pressure.

105. Figure 81 shows the magnitude of this bending moment M as a function of the void length L. It was drawn for the springline overburden pressure $p = 7800$ psf previously computed in Figure 41, for a lining modulus of elasticity $E = 6 \times 10^6$ psi and for a moment of inertia $I = 3.06 \times 10^7$ in.^{4} corresponding to a hollow circular cylinder with inner and outer diameters equal to the nominal values of 153 in. and 185 in. The effect of foundation modulus variation between $k = 200$ psi/in. and $k = 500$ psi/in., which bracket the stiffness of the material under the tunnel, is also illustrated in this figure.

106. This loading should be compared to the resistance of the concrete lining in longitudinal bending without axial thrust. This

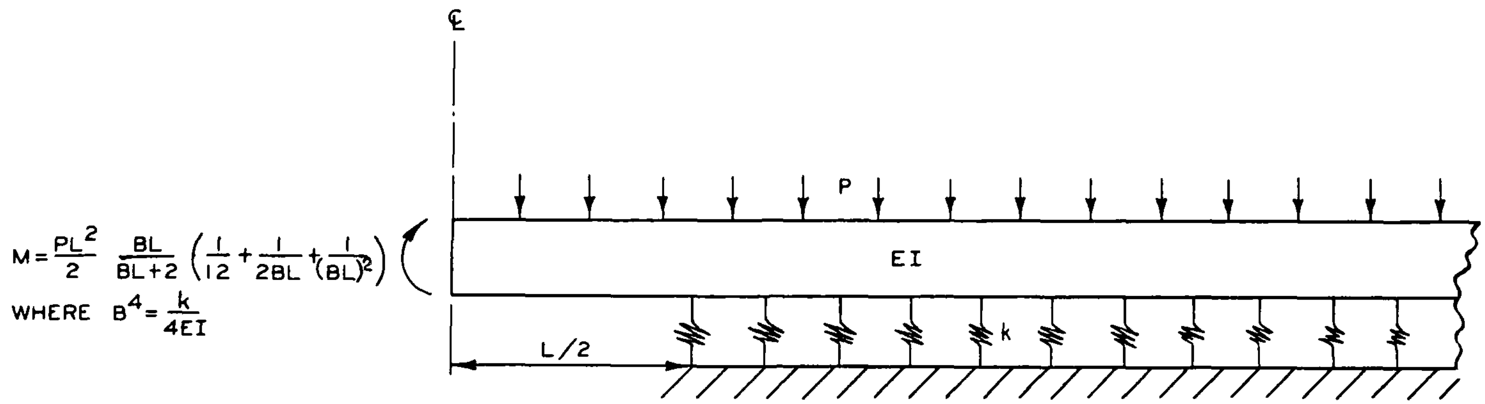


Figure 80. Longitudinal bending of concrete lining by loss of foundation support

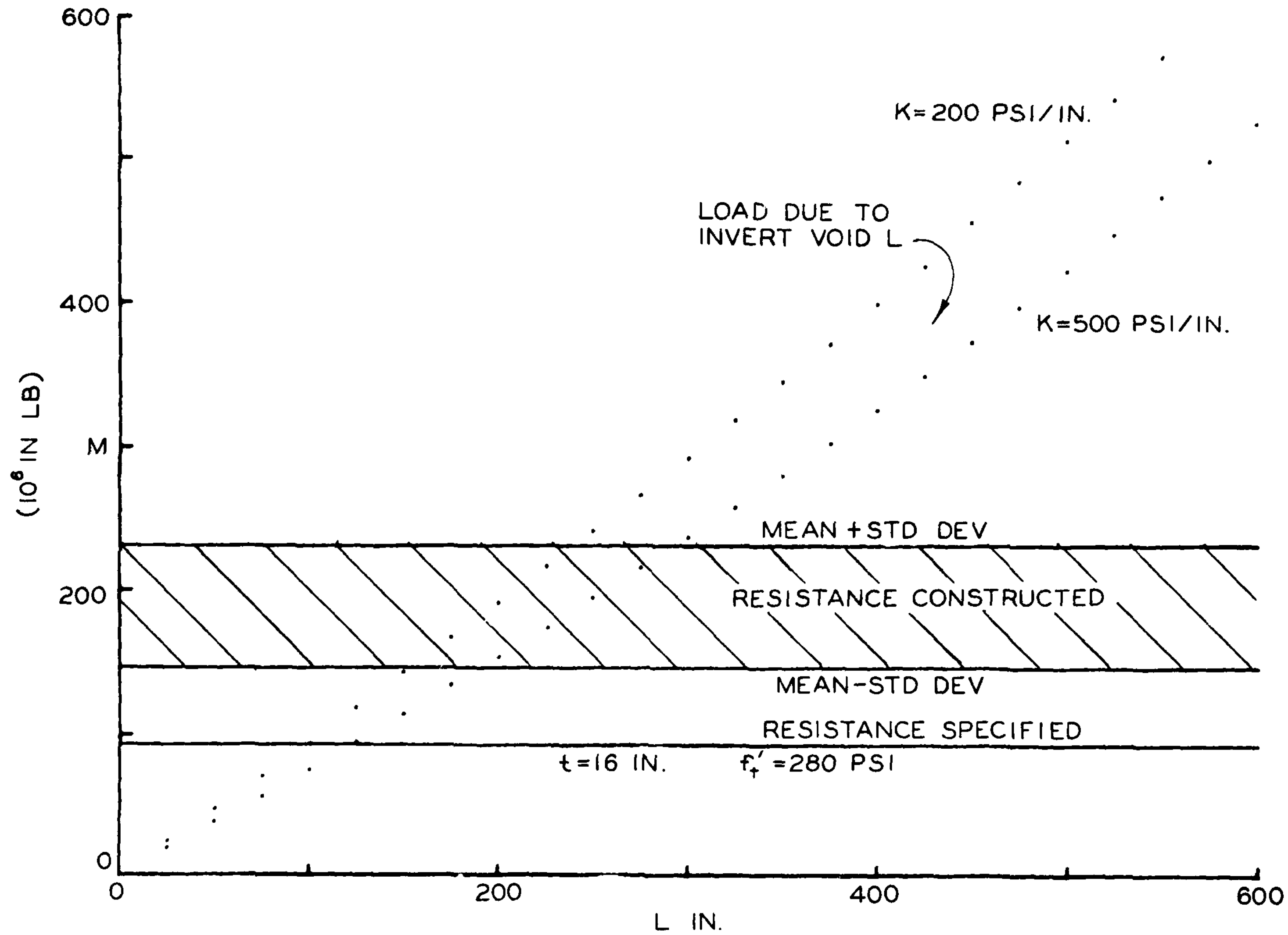


Figure 81. Reliability of concrete lining in longitudinal bending

resistance is approximated by the product of the concrete tensile strength and the elastic section modulus of the tunnel in longitudinal bending. Accordingly, for the contract-specified 16-in. wall thickness and concrete mixture discussed in paragraph 89, the resistance to longitudinal bending is approximately 93×10^6 in. lb as shown in Figure 81. The distribution of resistance in the lining as built depends on the distribution of wall thickness and tensile strength constructed. Using the Monte Carlo technique and the assumptions previously described in paragraph 91, it was estimated that the resistance of the majority of the concrete lining is contained within the hashed region of Figure 81. It is seen that the constructed lining has a resistance to longitudinal bending which generally exceeds that implicitly specified by the contract. It is further observed that the loss of foundation support due to invert piping for a longitudinal distance on the order of 200 in. could cause the tunnel to fail in longitudinal bending. This event would be accompanied by a circumferential cracking similar to that described in paragraph 73 near Distressed Area No. 3.

AD-A096 440 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY, DETROI--ETC(U)
JAN 81 D ALBERT, G C HOFF, B LORENCE NCE-IA-80-055
UNCLASSIFIED WES/TR/GL-81-2 NL

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PART IV: SUMMARY

Findings

107. Specific findings are presented sequentially as follows:

a. A note on the contract drawings states "Waterstops not required on this Contract." This refers to waterstops at construction joints.

b. During construction, Change Orders Nos. 2 and 4 were issued which provided for the installation of waterstops at construction joints not yet constructed. The construction records show that no waterstops were installed north of Sta 118+62.

c. The concrete placement lengths for the concrete liner were generally 105 ft. These substantial placement lengths directly increased the potential for cold joints in the concrete liner. This potential was frequently realized due to the late arrival of concrete trucks and plugged concrete drop pipes, resulting in pour delays.

d. The construction records show that numerous areas of the concrete liner required patching and repair of cold joints, honeycombing, and stone or rock pockets immediately following the removal of forms.

e. Stratification of the glacial lake deposits along the tunnel showed uniformly fine to medium graded beach sand unit along the base of the tunnel. In Distressed Areas Nos. 1 and 2, silt and sand strata also existed along the sides of the tunnel. Between Distressed Areas Nos. 2 and 3, silty to sandy clay existed along the sides of the tunnel above the sand along the base. In Distressed Area No. 3, thin strata of sand occurred in the silty clay along the sides of the tunnel and silt strata existed near the crown in the northern half of this section. On the basis of blow count comparisons, loose soil conditions were found in borings along all three distressed areas.

f. Special laboratory piping tests showed that fine to medium sands, silty sands, and silts from the site were susceptible to piping through small crack openings under water pressures of less than 2 psi, but the uniformly graded fine to medium sands were highly susceptible to piping under water pressures of less than 1 psi. Standard pinhole tests on silty clay indicated that this material was not susceptible to piping.

g. Groundwater conditions at the site in 1980 away from the area of dewatering, showed that the beach sand unit at the base of the tunnel had artesian water pressures that produced an equivalent groundwater level near the crown in Distressed Area No. 2 and up to 6 ft above the crown north and south of this area. These levels were more than sufficient for piping to occur.

h. Rainfall and river discharge data indicated 1976 to be the wettest year since construction, suggesting higher groundwater levels during 1976 than those found in 1980.

i. Staining in construction joints and from cold joints indicated leakage through these joints. Evidence of sand infiltration through construction joints and cold joints was found during field work.

j. Significant seismic activity did not occur from 1972 to 1980.

k. The concrete composition, homogeneity, and quality were satisfactory. No significant differences were found in the concrete between distressed and intact areas.

l. Seepage, honeycombing, and contamination occurred in continuous cold joints in the side walls of both the distressed and intact areas.

m. The distressed areas are characterized by longitudinal cracking near the crown and invert, longitudinal spalling near the springlines, a contraction of crown-invert diameter, and an expansion of springline diameter.

n. The loading condition assumed in the original design of the concrete tunnel wall was a uniformly distributed external pressure equal to the estimated weight per unit area of the soil between the ground surface and the springline.

o. The specifications in the contract provided for a resistance which exceeded the assumed uniform design loading.

p. The as-built structure had a resistance against external loading which exceeds that specified in the contract.

Conclusions

108. Conclusions are presented sequentially as follows:

a. Seismic activity was not a factor.

b. Concrete composition, homogeneity, and quality were not factors and there was no evidence of detrimental chemical reactions.

c. At end of construction, the concrete liner contained open construction joints and/or cold joints at several locations.

d. The fine to medium sands, silty sands, and silts will pipe through openings as narrow as 0.01 in. under water pressures less than 2 psi.

e. After construction, the external water pressure on the tunnel invert ranged from 4 to 8 psi.

f. Soil piped into the tunnel in varying amounts depending on the location of the strata of piping soil with respect to open construction joints and/or cold joints. As material piped into the tunnel, the tunnel lost bottom and side support.

g. When loss of support occurred beneath the invert, the resulting loading caused the structure to crack circumferentially as was observed in Distressed Area No. 3.

h. When loss of support occurred at the springline, the resulting nonuniform loading caused the resistance of the structure to be exceeded and initiated the pattern of ovalling and longitudinal cracking and spalling observed in all distressed areas.

109. In summary, the distress actually began immediately following construction as soon as the groundwater level was sufficiently high to initiate piping of soil through open construction and cold joints. This piping took place over a significant period of time. As greater loss of support occurred, the concrete liner deformed, the cracks opened wider, and more soil was allowed to pipe into the tunnel. These events progressed until the distress was manifested by the crack pattern found, and by total collapse at Distressed Area No. 1 and partial collapse at Distressed Area No. 3. Varying degrees of distress were experienced depending upon the location of the strata of piping soil with respect to open construction and/or cold joints.

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Table 1

Excavation Summary

Date	Start Sta.	Stop Sta.	Mined (feet)	Ribs (#)	Soil	Construction Problems and Significant Events
10-30-71	135+86	135+83	6	2		
11-02-71	135+83	135+79	4	NS	Clay w/sand	
11-08-71	135+47	135+43	4	NS	Hard grey plastic clay w/rocks and sand	Grade is .50' high. Top Jack Not working.
11-09-71	135+43	135+35	8	2	"	
11-10-71	135+35	135+23	12	3	Firm to soft gray plastic w/sand and rocks	
11-11-71	135+23	135+07	16	4	Firm to soft gray clay w/sand and rocks	
11-12-71	135+07	135+95	12	3	"	Conveyor belt giving trouble.
11-13-71	134+95	None	0	0		Repair conveyor belt.
11-15-71	134+95	134+83	12	3	Firm to soft gray clay w/some rocks	
11-16-71	134+83	134+63	20	5	Plastic gray clay w/small stones	
11-17-71	134+63	134+35	28	7	Firm to soft plastic like clay	PI #5 is 2.10'W. PI #6 is 2.20'W. Rib .431 is 1.8' high and Rib #438 is 1.2' high.
11-18-71	134+35	134+19	16	4	Gravel and sand w/firm gray clay and rocks	PI #7 is 2.40'W PI #8 is 2.10'W. Rib #42 is .56' high. Sand in rollers. Top of Mining is gray clay. Face is 75% dry sand. Cavity 4.0' ahead of machine.
11-19-71	134+19	133+99	20	6	Gravel and sand w/firm gray clay	Rib #46 is .46' high. Rib #50 is .70'E.

Table 1 (continued)

11-19-71	133+99	133+83	16	4	Gravel-sand w/gray clay	Rib #53 is .20' low and 1.40'E Incline on machine is .27' low.
11-23-71[1]	133+83	133+63	20	5	Sand and gravel w/gray clay	Rib #9 is .14' low. Can't check line.
11-24-71[1]	133+63	133+31	32	8		
11-26-71[1]	133+31	None	0	0		Remove sand and debris from heading.
11-27-71[1]	133+31	None	0	0		Remove secondary conveyor used in curve.
11-28-71[1]	133+31	None	0	0		Place sled and conveyor.
11-30-71[1]	133+31	None	0	0		Place hydraulic system and conveyors.
12-01-71[1]	133+31	None	0	0		Hydraulic, electrical and conveyor system assembly troubles.
12-02-71[1]	133+31	None	0	0		"
12-03-71[1]	133+31	None	0	0		"
12-04-71[1]	133+31	None	0	0		"
12-06-71[1]	133+31	133+35	4	NS	Very hard grey sand w/stones and clay	Cable between machine and sled broke.
12-07-71[1]	133+35	133+20	15	3	Very hard gray sandy clay w/stone and gravel vein in bottom	Rocks and hydraulic system problems.
12-07-71[1]	133+20	132+95	20	5	Hard gray clay w/stone	Rib #73 is .60' high and .80'E. Rib #75 is .56 high and .75'E. At both ribs machine is inclined .05' looking west.
12-08-71[1]	132+95	132+85	10	2	Very hard gray sandy clay w/stones	Failure of hydraulic system. Rocks - 2' in diameter.
12-08-71[1]	132+85	132+70	15	3	Hard gray clay	Rib #80 is .45' high. Laser lite hitting carrier.

Table 1 (continued)

12-09-71[1]	132+70	132+50	20	4	Gray sand w/very hard	Hoisting crane down and mud cars off track.
12-09-71[1]	132+50	132+25	25	5	Gray sand & clay w/rocks	Problems with rocks, clean up, electrical relay, hydraulics and mud car derailment. Grade is .30' high. machine is moving to the west.
12-10-71[1]	132+25	132+05	20	4		Soft wood lagging broke - (3 pieces). Hoisting crane will not start. Rib #93 is 1.42'W and .14 high.
12-10-71[1]	132+05	131+80	25	5	Same	Distance from correct alignment at ribs #91 is 1.0', #92 is .87', #93 is .68', #94 is .58', #95 is .33', #96 is .28', #97 is .15' and #98 is .00'. Rock, hand mining and hydraulic problems. Machine still mining to west, some roll corrected.
12-11-71[1]	131+80	131+65	15	3	Hard sandy gray clay w/stones	Hydraulic system problems. Rib #101 is 1.75'W.
12-11-71[1]	None					Mining machine repairs.
12-13-71[1]	131+65	131+60	5	1		Add to conveyor.
12-13-71[1]				1	Big boulder	Hydraulic system probelms.
12-14-71[1]	131+55	131+25	30	6	Very hard gray clay w/stones at bottom Stoney soft clay above springline	Rocks, teeth and alignment problems.
12-14-71[1]	131+25	131+05	20	4	Hard clay to springline & sandy clay on top	Rock, conveyor and dinkey problems. Machine moving to east.
12-15-71[1]	131+05	130+90	15	3	Very hard gray sandy clay w/stones	Mining machine's rear can developed radial crack from springline to crown. Machine grade is 15.' high.
12-15-71[1]	130+90	130+65	25	5	Sand gray clay. Dry	Rocks, and mud car derailment problems.

Table 1 (continued)

12-16-71	130+65	130+35	30	6	Hard gray sandy clay w/stones	Slow dinkies. Machine is .50'W.
12-16-71	130+35	130+25	10	2		Machine coming on to line. Grade is .20'± high. Dinky broke. Fan electrical line cut.
12-17-71	130+25	130+00	25	4	"	Electrial, track, derailment and rock problems.
12-17-71	130+00	129+90	10	1		Dinky problems. Line and grade good.
12-18-71	129+90	None	0	0		Power supply cut.
12-18-71	129+91	129+56	35	7	Same	Rock and conveyor problems.
12-20-71	129+56	129+16	40	8	Very hard gray clay w/hard sandy spots w/stones	Sled, conveyor, hydraulic and rock problems.
12-20-71	129+16	128+76	40	8	Rocky gray clay & sand	Rock problems.
12-21-71	128+76	128+26	50	10	Gray sand w/firm gray clay w/stones	Rock problems.
12-21-71[2]	128+26	127+76	50	10	Same	Spilled ribs & lag.
12-22-71[2]	127+26	127+56	20	4	Clay very rocky	Shot boulder. Hydraulic, conveyor and rock problems.
12-22-71[2]	127+56	127+21	35	7	Same	Rock and teeth problems.
12-23-71[2]	127+21	126+96	25	6	Very rocky, sandy gray clay, large boulders.	Blasting rocks.
12-23-71[2]	126+96	126+76	20	4	Clay & rock	Rock and mining machine problems. Line is 0.35W.
12-23-71[2]	126+76	126+61	15	2	Very hard gray sandy clay w/traces of soft clay in crown & stones	Conveyor problems. Machine grade is .15' high.

Table 1 (continued)

12-27-71[2]	126+61	126+21	40	8	Same	Rock problems. Machine is .30' low. Line is .10'E."
12-27-71[2]	126+21	125+81	40	8	Sandy clay & rock	Blasting rock. Machine is .25' low. Derailment, and clean up problems. Line is .10'W. Grade is .20' high.
12-28-71	125+81	125+36	45	9	Hard clay & sand w/rocks some water.	Blasting and removal of large rock. Machine is .05' high.
12-28-71	125+36	125+01	35	7	Clay & rocks	Shield for mining machine split more on East side. Dinky (train) problems. Line is .20' East.
12-29-71	125+01	124+81	20	5	Same	Hydraulic problems.
12-29-71	124+81	124+41	40	8	Clay & large rock	Blasting rock. Line is East. Grade is obstructed but coming out.
12-30-71	124+41	124+11	30	6	Same	Rock and conveyor problems. Machine and conveyor rolled. Laser still not clear.
12-30-71	124+11	123+76	35	7	Large rock & clay	Machine back on line. Blasted rock. Line is .06' E.
12-31-71	123+76	123+46	30	6	Very hard gray clay w/stones & damp. at top	Hydraulic problems.
01-03-72	123+46	123-01	45	9	Gray clay & sand	Small problems. Ribs #190 and #210 shims fell out.
01-03-72	123+01	122+71	30	7	Same	Mud car derailment and tooth problems. Blasted rocks. Replace shims at ribs #190 and #210. Machine is .30'E & .15 low.
01-04-72	122+71	122+31	40	8	Same	
01-04-72	122+31	122+16	15	3	Hard & full of rocks	Line is .20' East. Conveyor problems. Blasting rock.
01-05-72	122+16	121+86	30	6	Very hard rock gray sandy clay	Rock problems.

Table 1 (continued)

01-05-72	121+86	None				Rib jack, conveyor and rib jacking frame problems.
01-06-72	121+86	121+51	35	7	Firm gray sandy clay w/stones	Rock, rib jack and dinky problems. Grade is .07' high to .04' high.
01-06-72	121+51	121+21	30	6	Sandy clay w/rocks	Blasting rock. Grade good. Line good.
01-07-72	121+21	120+96	25	5	Very hard gray clay w/large rocks	Problems with large rock. Grade is .05' to .00' high. Line is .19'E to .24'E.
01-08-72	120+76	None	0	0		Broke tail section of machine.
01-08-72	120-76	120+61	15	3	Very hard & rocky clay some sand showing on top	Grade is .35' high. Excessive jacking pressures required from tight mining. Dinky problems. Skin was welded.
0108-72	120+61	120+46	15	3		Grade is .32' high to .24' high.
01-10-72	120+46	120+06	40	8	Very hard gray sandy clay w/stones	Thirty inch diameter hole drilled at 120+46 ± for ventillation. Rock problems. Rib grade is .23' high to .00'. Line is .05'W to .00".
01-10-72	120+06	119+91	15	3	Rocky	Hydraulic system, mud car derailment and cut electrical power problems.
01-11-72	119+91	None	0	0		No machine operator. Clean up mud from mud car derailment.
01-11-71	119+91	119+26	65	13	hard gray clay & rockly	Blasting rock. Mud car derailment Line is OK.
01-12-72	119+26	118+81	45	9	Very hard gray sandy clay w/stones	Rocks, dinky and fan line problems. Grade is .10' low to .05' low.

Table 1 (continued)

01-12-72	118+81	118+56	25	5	Hard gray clay	Mud car derailments (2), and dinky problems. Blasting rocks. Grade is .10'high. Line is .20'E.
01-13-72	118+56	118+21	35	7	Very hard gray stoney clay w/soft clay at top	Dinky, rock hydraulic and grubber clamp problems. Grade is .01' high to .05' high. Line is .10'E to .05'E.
01-14-72	118+21	117+81	40	8	Extremely hard sandy	Mechancial and rock problems. Grade is .06' high to .03' high. Line is .06'E to .03'E.
01-14-72	117+81	117+46	35	7	Hard clay & rocks	Rocks problems.
01-15-72	117+46	117+06	40	8	Very hard gray sandy clay	Rock and mechanical problems. Machine is .30' low.
01-15-72	117+06	116+81	25	5	Sand on bottom. Hard gray clay w/rocks	Conveyor and rock problems. Grade is .05' low. Line is .10'E.
01-17-72	116+81	116+41	40	8	Very firm gray sandy clay w/sandy bottom & stones	Rock, mechanical and dinky problems. Grade is .06' low to .00'. Line is .05'W to .00'.
01-17-72	116+41	116+01	40	8	Hard gray clay w/boulders	Mechanical, track, electrical and rock problems. Line and grade are good.
01-18-72	116+01	115+76	25	5	Very hard gray rocky clay with a sand seam throughout drive	Hydraulic, mechanical and rock problems. Grade is .22' low to on grade. Line is .04E to .21E.
01-18-72	115+76	115+41	35	7	Hard gray clay w/large rocks	Toothholder, conveyor and rock problems. Grade is .10' high to .00'. Line is .15'E to .00'. At rib #312, lagging board hanging at top (Set on 1-7-72).
01-19-72	115+41	None	0	0		Two rocks, motor on machine and power supply problems.
01-19-72[3]	115+41	115+11	30	7	Clay/sand on bottom	Hydraulic and conveyor problems. Roll in power trailer.

Table 1 (continued)

01-20-72[3]	115+11	114+86	25	5	Very hard gray clay w/large stones	Rock and conveyor problems. Two motors damaged on mining machine. No operator. Grade is 07' high to .18' high. Line is .03'E to .09'W.
01-20-72[3]	114+86	114+66	20	4	Hard gray clay & rocks w/sand	Radial hydraulic motor removed. Hydraulic problems. Blasted rocks. Grade is 05' low.
01-21-72[3]	114+66	114+36	30	6	Very hard gray clay w/sand seam in bottom w/stones	Hard ground, conveyor and rock problems. Rib #9 is pushed out. Grade is .10' high to .05' high. Line is OK.
01-21-72[3]	114+36	114+06	31	NS		
01-22-72[3]	114+06	113+71	35	7	Firm gray sandy clay w/stones, dampness in bottom	Blasted rocks. Grade is .08' high to .04' high.
01-22-72[3]	113+71	113+31	40	8	Hard clay w/rocks sandy layer at springline	Rock problems. Grade is OK. Line is .04'W to .25'E.
01-24-72[3]	113+31	113+11	20	4	Hard gray clay w/sandy bottom and top & stones	Dinky, conveyor, electrical and rock problems. Grade is .17' high. Line is .31'W to .02'W.
01-24-72[3]	113+11	112+76	35	7	Rocky, hard clay with traces of sand, small amount H_2O.	Hydraulic problems. Grade is .17' high to .02' high. Line is .05'W to .00'.
01-25-72[3]	112+76	112+46	30	6	Firm gray sandy clay w/stones	Rock, conveyor and hydraulic problems. Line is .15'W to .09'W. Grade is .12' low to .02' high .
01+25+72[3]	112+46	112+16	30	7		Rock and conveyor problems. Line is OK. Grade is .25' high.
01-26-72[3]	112+16	111+91	25	5	Hard gray clay w/stones	Rock, conveyor and electrical problems. Line is .15'W to .00'. Grade is .17' high to .00'.
01-26-72[3]	111+91	111+56	35	7	Sand layer at springline. Hard clay w/rocks	Conveyor, dinky and rock problems. Grade is .15' high.

Table 1 (continued)

01-27-72	111+56	111+31	25	5	Firm gray clay w/large stones. Signs of water in bottom	Conveyor and rock problems. Grade is .03' high. Line is .00' to .05'E.
01--27-72	111+31	111+01	30	6	Hard gray clay	Conveyor and hydraulic problems. Blasting rock. Line is .15'W to .05'W. Grade is .08' high to .00'.
01-28-72	111+01	110+71	30	6	Hard gray clay w/sandy wet bottom & stones	Rocks blasted 4 times. Mud car derailment problem. Grade is .04' low to .09' low.
01-28-72	110+71	110+41	30	6		Rocks and mechanical problems.
01-29-72	110+41	110+16	25	4	Hard ground	Hard ground problems. Rocks blasted 6 times. Grade is .04' low to .02' low. Line is .10'W to .02'E.
01-29-72	110+16	110+01	15	3	Hard clay	Lack of cutting edge. Blasting rocks. Grade is .10' high.
01-31-72	110+01	109+71	30	6	Very hard gray clay w/large rocks	Rock and mechanical problems. Grade is .00' to .18' high. Line is .05'W to .18'W.
01-31-72	109+71	109+36	35	7	Hard clay & stones	Rock, hard ground and motor problems. Blasted rocks. Grade is .15' high to .01' high. Line is .19'W to .24'W.
02-01-72	109+36	109+16	20	4	Very soft mushy clay w/stones	Sand above last two ribs caused a void approximately 5' above machine. Grade is .01' high. Line is .19'W to .21'E.
02-01-72	109+16	108+93	23	NS	Sand & gravel. Little clay.	Face slipping in eight foot over mine on crown. Face is on a slope out 8 to 10 feet from the machine. Void in the crown starting at Station 109+26. Line is .26'E to .46'E.
02-02-72	108+93	108+88	5	2	Adverse soil, w/water, sand, silt, some stones	Sand above machine is still falling, causing an approximate 8' void above machine. Conveyor problems. Machine is .02' low and .04'W.

Table 1 (continued)

02-02-72	108+88	108+86	2	0		Conveyor problems. Mud is stickey, hard to work and won't come out of machine. Water running in sides clear. Void in crown is 15' above mining machine.
02-03-72	108+86	108+85	3	1	Loose soil, water & plastic clay	Cave in above machine which progressed to surface. Void at surface was approx. 35' in diameter and 20' deep. It was backfilled.
02-03-72	None	None	0	0		Dug sump hole in east side in invert. Reinforced ribs are taking load. Water pump working good down hole.
02-04-72	108+85	108+78	7	3	Sand, silt, water with some gray clay.	Mud along tracks and conveyor problems.
02-04-72	108+78	108+63	15	7		Conveyor and dinky motor problems. Water coming in fast around bottom shield. Face still falling in on face of machine. Closing holes in wheel. Pumped water out down hole.
02-05-72	108+63	108+58	5	2	Sand, gravel and water and traces of clay	Conveyor problems.
02-05-72	108+58	108+43	5	6	Sandy with hard pan gravel	Pumping water out down hole.
02-07-72	108+43	108+36	13	5	Entire face is sand mixed with coarse sand & gravel in bottom and water entering heading	Conveyor problems. Sand and silt around sleds is causing excess pressure on hydraulic system. Operator is attempting to carry machine .20' high to compensate for possible rib deflection in crown.
02-07-72	108+36	108+31	5	2	Looking better	Conveyor belt problems. Conveyor and mole separated. Pumping water out top hole.

Table 1 (continued)

02-08-72	None					Clean up silt and mud from around sleds, tracks and conveyor. Water continues to enter heading at and behind machine. Contractor now using two pumps to transport water up alignment hole and out of heading.
02-08-72	108+31	108+11	20	9		Face still spalling in 6' to 8' on crown and face.
02-09-72	108+11	107+78.5	32.5	14	Loose sand with some sharp sand and gravel also chunks of clay which appear to be falling from above machine	Void above machine, with signs of settlement clear to surface.
02-09-72	107+78.5	108+65.5	15	NS		Crown coming in. Large void at surface. Crack from void over top of gas pipe. Stopped mining.
02-10-72	None					Face of mining falling in. Mining stopped. Surface settlement. More openings covered on wheel to prevent sand from coming in.
02-11-72	None					Plugging holes in lagging to prevent water and silt from coming in.
02-16-72	None					Test holes drilled 5' in front of machine wheel. Hole at 5' West of cylinder to 62' which showed firm clay. Hole on centerline showed firm clay to a point 5' above machine, from this point ot approx. 1' above concrete invert showed white sand with slight traces of clay.
02-16-72	None					DMWS recommend last 5 ribs be reinforced because of overloading.

Table 1 (continued)

02-17-72	None	Ribs started to take weight and deflected. Three jump sets were placed and welded. Holes where water coming in were plugged.
02-18-72	None	
02-21-72	None	Checked lagging for leaks and checked pumps.
02-22-72	None	Packing straw in and place plywood over cracks between lagging where slit and water entered heading through crown.
02-23-72	None	Closing Red Run north of cofferdam. Water entering cofferdam at SE corner.
02-24-72	None	Plugging space between lagging through wet area where water and silt enter into heading. Pumping water and trying to plug hole at SE corner of cofferdam.
02-25-72	None	Pour retaining wall in cofferdam.
02-28-72	None	Heading OK - per Mr. Jim Heath.
02-29-72	None	Retaining wall resteel OK. Water entered cofferdam and stopped work.
03-01-72	None	At cofferdam a bulkhead was being installed to prevent water from entering the tunnel south of Red Run.
03-01-72	None	Add 6" electrical pump in cofferdam.
03-01-72	None	Remove silt and sand from cofferdam.
03-03-72	None	Some settlement at cave in area approx. 3 cubic yards loss. However, there is no sign of a loss of ground in heading.

Table 1 (continued)

03-07-72	None	No further settlement at cave in area.
03-09-72	None	Remining started where machine was 2.20' high. Ribs through caved in section showed no further signs of deflection.
03-10-72	None	Plugged some cracks between lagging in heading to prevent silt from filtering through.
03-13-72	None	In the remined section. 32' of lagging boards from invert up to springline were removed.
03-14-72	None	Plugging cracks between lagging where water and silt coming into heading.
03-15-72	None	Remining continued. Seven ribs were cut, lowered and rewelded so that 12" concrete with resteel can be used.
03-16-72	None	No further deflection on ribs throughout caved in area. Sheeting and wales in cofferdam showed deflection due to excessively heavy ground around bracing.
03-17-72	None	Extra bracing placed in cofferdam.
03-18-72	None	At cofferdam knee braces placed and welded on three top wales at North end.
03-20-72	None	At the ribs in the heading, no further deflection other than that caused by initial over loading. Removed from cofferdam silt and washed in during flooding.
03-22-72	None	Ribs throughout caved in area had not deflected any more than on 02-12-72.

Table 1 (continued)

03-23-72	None				Bottom now changed from sand to hard sandy gray clay.	Remined section South of Station 136+00 in curve.
03-24-72	None					Cutting, lowering and rewelding 3 ribs.
03-25-72	None				Bottom is very hard sandy gray clay w/stones	Removed 3 ribs at springline, remined bottom and replaced ribs to accomodate 16" concrete.
03-28-72	None					Removed 5 ribs, remined to planned grade and replace ribs by welding in place.
03-30-72	None					First pour of concrete at 135+82 to 135+67.
04-08-72	None					Removed boards at west springline 134+92 to 134+12.
04-21-72	None					Downtime due to malfunction of hydraulic system on form carrier.
04-25-72	None					Took shots on ribs at cave in area and found no additional settlement.
05-04-72	None					Carrier motor problem.
05-06-72	None					Plugged space around alignment pipe at Station 108+26 where approx. 8 cubic yards of silt and sand entered heading over night. Line check not made because of silt and sand blocking tunnel.
05-08-72	107+65	107+63	2.5	1	Wet sand mixed with traces of wet clay	Clean up silt and sand at Station 108+26. Track and electrical problems.
05-08-72	107+63	107+58	5	1	Very wet mud & sand	Conveyor problems. Muck coming in is slowing down conveyors and stopping them. More holes closed in wheel. Overexcavated 20 mud boxes (about 49 cu. yards.).

Table 1 (continued)

05-09-72	107+58	107+48	10	4	Mixed clay, sand, gravel & water, firm silty clay bottom of mining to 4 foot above, sand and gravel to top quarters with dry sand above top quarters.	More holes on wheel were closed. Overexcavated 7 mud boxes (about 17-1/2 cubic yards.)
05-09-72	107+48	107+38	10	4	Soft gray clay mixed with sand, gravel, & water	Ground stable. Void in front of mining machine approx. 9' wide, 4' high and 3' ahead of machine. Over excavated about 12 cubic yards. Blasted rock.
05-09-72	107+38	107+25.5	12.5	5	Moist sand (hardpan)	Mud car off track. Over excavated about 12.5 cubic yards. No ground settlement.
05-10-72	107+25.5	107+09	14.5	6	Full face of clay, very hard with considerable amount of rocks throughout	No excess mining. Problem with rocks. Machine is .80'W and .29' high.
05-11-72	107+09	106+71.5	27.5	11	Very hard gray clay w/many rocks througout	Grubbler tooth, rock and mud car derailment problems.
05-12-72	106+71.5	106+58	22.5	5		Rib #643 is .15' high and .75'W. Compressor, hydraulic, conveyor and mud box problems.
05-12-72	106+58	106+38	20	4	Hard gray clay w/scattered stones	Sand and silt infiltrating through lagging on east side throughout day. Machine is out .65'W and .28' high.
05-12-72	106+38	106+18	20	5	Hard gray clay w/many rocks.	Machine is .60'W and .40' high. Mud car derailmnet problems.
05-12-72	106+18	106+03	15	3	"	Machine is .45' high and .35'W. Wheel hung up. Jack broken. Rocks were spaded out.
05-13-72	106+03	105+88	15	3	Very hard gray clay w/stones and some wet spots in top quarters and crown.	Machine at end of 3rd rib was .15'W and .25' high. Mud car derailment and mechanical problems.

Table 1 (continued)

05-13-72	105+88	105+78	10	2	Mostly sand and small amount of clay and then hard gray clay mixed with rocks & sand	Void above rib #662 approx. 7' high and 12' wide. Mud car derailment and hydraulic problems. Mining machine to .15' high and .05'W.
05-13-72	105+78	105+68	10	2	Dry sand with some clay w/stone throughout	Removed 23 boxes of mud for each rib (over excavated about 45 cubic yards total). Blasted rock.
05-15-72	None					Loss of ground above machine. Large wheel holes on mining machine were closed.
05-15-72	105+68	105+58	10	4	Hard gray clay, mixed with sand & rocks throughout.	Rock problems. Over excavated about 37 cubic yards of clay.
05-15-72	105+58	105+44.5	13.5	4	Hard gray clay mixed w/sand & rock	Rib #670 is .15' high and .12'E. Wheel hund up. Blasted rock.
05-16-72	105+44	105+33	11.5	4	Very hard gray sandy clay	No excess earth removed. Blasted rocks. Conveyor problems. At 4th rib machine was .15'W.
05-16-72	105+33	105+12	21	7	Very hard gray clay w/traces of sand & many rocks.	Machine is .30'W and on grade. Rock problems.
05-16-72	105+12	104+97	15	6	Hard gray clay with stones and sand throughout	Machine is .25'W and .05' high. No excessive mud. Top of ground over tunnel holding. Conveyor problems.
05-17-72	104+97	104+89.5	7.5	3	Very hard gray clay w/scattered stones, 1.5' of top of mining is wet sand and silt.	No loss of ground. Rock, hydraulic and compressor problems. At rib #693 machine was .50'W and on grade.
05-17-72	104+89.5	104+72	17.5	7	Hard gray clay w/many rocks throughout. Wet silty sand in crown.	No excessive mud boxes. Machine is .20' high and .15'W.
05-17-72	104+72	104+60.5	15	6	Soft clay & sand in top w/hard pan in bottom w/rock.	Corrected stationing from 104+57 to 104+60.5. Machine is .05'E and .25'. Blasted rock. Compressor and dinky problems.

Table 1 (continued)

05-18-72	104+60.53	104+55.53	5	2	Very hard gray clay approx. 3' above spring-line w/silt & sand above machine	Void approx. 6' above machine on SE side. Over excavated approximately 30 cubic yards. Wheel was plugged from falling earth.
05-18-72	104+55.53	104+28.03	22.5	9	Hard gray clay some clay w/considerable amount of rocks throughout. Crown shows silty sand & water.	Over excavation from rib #709 to #711. Machine is .10'E and .15' high. Blasting rock. Rocks were spaded.
05-18-72	104+28.03	104+18.03	10'	4	Very wet running in top, soft clay and sand with stone throughout in bottom	Machine is .15' high and .20'E. Void in crown. At ground level the ground is holding . No significant signs of ground moving. Overmined 60 cubic yards.
05-19-72	104+28	104+15.5	2.5	1	Face of mining was firm gray clay w/silt running into wheel from top west quarter.	10 cubic yrds overmined.
05-19-72	104+15	None				A grout hole was drilled at approx. Station 10.+35. It was decided to fill the void at a later date with grout or concrete.
05-20-72	None					At Station 104+30 a grout hole drilled approx. 4' East of the centerline showed a void 30' deep which was grouted. At Station 104+30 a grout hole drilled approx. 4' West of the centerline showed no void. At Station 104+25 a grout hole drilled 3' west of the centerline showed a void. The void was filled. At Station 104+10 a grout hole was drilled and a void approx. 7' above the machine was found and filled. Forty cubic yards of pea gravel was used. One half a cubic yard of grout came through wheel.
05-22-72	None				Soft ground w/silt water & rocks	Eight boxes of mud for a 6 inch shove. (Over excavated about 16 cubic yards)

Table 1 (continued)

05-22-72	None					Wheel plugged up. Check at ground level showed ground holding. Cover plates were placed over the wheel openings.
05-22-72	None					Silt and sand infiltrating into heading through lagging. Electrical problems.
05-23-72	104+15.5	104.03	12.5	5	Very soft mud like clay mixed w/silt and water	Machine is .25'E and .45' high. Void in crown approx. 12' ahead of the machine 10' high and 12' from the centerline toward East of tunnel. Check at ground level showed ground holding. Removed 14 mud boxes for each rib. (Over excavated about 82 cubic yards total)
05-23-72	104+03	103+93	10	4	Very soft mud & sand mixed with rock	Machine is .20'E and .40' high. At ground level the ground is holding. Over excavated about 60 cubic yards. Mud cars off track.
05-24-72	103+98 per report	103+83.5	14.5	5	Silty ground. At sixth rib very hard gray clay	Excessive infiltration. Eighteen cubic yards overmined. A void of 90 cubic yards occured at ground surface at station 104+20. No sign of ribs and lagging in heading taking excessive load.
05-24-72	103+85.5 per report	103+65.5	20	8	Hard to firm gray clay w/rocks	Full face is holding in front of machine. Machine is .05'E and .05' high.
05-24-72	103+65.5	103+63	2.5	1	Hard & firm gray clay w/rock	Mining wheel hit steel sheeting at cofferdam.
05-30-72	119+67	South				At approximately 118+57 a drop hole was drilled through a field tile. Water came in "very fast" with clay mixed in. Tried to stop and/or plug it. Got it plugged up but then it

Table 1 (continued)

			failed, all the fill used in plug ended up in the tunnel. Lagging was put back in and pump started. Approx. 8 mud boxes in tunnel. (About 20 cubic yards) Water entered just as the concrete pour was completed up to Station 119+67.
06-05-72	115+47	114+27	Cleaned up mud and water.
06-06-72	114+42	113+22	Cleaned up mud and water.
06-07-72	113+37	112+17	Cleaned up mud and water.
06-08-72	112+32	111+12	Cleaned up mud and water.
06-09-72	111+27	110+19	Cleaned up mud and water.
06-10-72	110+22	109+17	Cleaned up mud and water.
06-12-72	109+17	108+07	Cleaned up mud and water. Electrician called in to start up pumps. Weather was rain.
06-19-72	104+97	103+92	Cleaned up invert. Removed boards that were sticking up in invert.
06-20-72	111+27	South	Finishers cannot finish invert because of standing water.
06-22-72	Cofferdam	North	Crew climbed out of shaft at Red Run and used 10 cubic yards of concrete to stabilize bottom where sand was bubbling in. (Concrete tunnel lining was up to 103+47 on this date . No notes as to cofferdam sheeting being cut.)
06-23-72	Cofferdam		Bottom of tunnel in cofferdam was poured up to the springline.

Table 1 (concluded)

06-28-72	135+65	120+71		Grout holes between these stations were drilled and only the concrete at Station 135+68 was not tight to lagging.
06-29-72	126+12	111+26		Grout holes between these stations were drilled and only the concrete at Station 126+12 was not tight to lagging, and a void in the crown was noted.
07-05-72	108+17	103+91		Grout holes between these stations were drilled. All concrete was tight to lagging.
07-07-72	126+12			Filled void at Station 126+12.
07-07-72	12+42			Filled power drop hole.
07-14-72	Cofferdam			Backfilled 3 feet over tunnel.
08-03-72	Cofferdam			Removal of sheeting was started.
08-16-72	Cofferdam			During the night the river came in and filled the shaft to the level of the river.

1. Represents work in distressed area number 1.

2. Represents work in distressed area number 2.

3. Represents work in distressed area number 3.

4. <u>NS</u> under the column <u>Ribs</u> means that the number of ribs set that shift were not specified in the inspector's reports.

Table 2

Concrete Summary

Date	Concrete Placed		Time		Operation/Problem/Notes
	From	To	From	To	
3-31-72	135+82	135+67	1300	1445	Pour//Resteel in lower east quarter from station 135+67 to 135+82.
4-05-72			0700	1330	Form
4-06-72	135+67	134+92	1645	2145	Pour/Void in arch at bulkhead 31 high and back approx. 32'/Restee at lower west quarter at station 135+52
4-08-72			0700	1530	Form
4-10-72	134+92	134+12	1820	2345	Pour/Void over old work may only be partly filled/Resteel at west springline from Station 134+87 to 134+42. Resteel at invert from Station 134+12 to 134+27
4-11-72					Strip//Section fair
4-12-72				2400	Strip/Stone pockets from west quarter point to east quarter point Station 134+97
4-13-72[1]	134+12	133+32	1230	1615	Pour 80'. Stripped bulkhead with 15' arch and invert. Form 15'//Resteel at west springline at Station 133+82
4-14-72[1]					Strip/Rock pockets at Station 134+12
4-14-72[1]			0700	1500	Strip/Rock pocket 3' Dia. west springline
4-14-72[1]			1600	2400	Form 15' arch and invert. Strip 15" arch.
4-15-72[1]			0700	2400	Strip and form. Sixty feet formed

1. Represents work in distressed area number 1.
2. Represents work in distressed area number 2.
3. Represents work in distressed area number 3.

Table 2 (continued)

Date	Concrete Placed		Time		Operation/Problem/Notes
	From	To	From	To	
4-17-72[1]	133+32	132+27	1125	1650	Pour/Pour progressed very slowly due to slow arrival of concrete trucks./Resteel in bottom 1/2 from Station 132+57 to 133+17
4-20-72[1]					Strip/Honeycomb at springline east and west Station 133+32
4-20-72[1]			1600	0100	Form 35' of invert and remove bolts for next 35'
4-21-72[1]			Day	SHift	Strip and form 70' each
4-21-72[1]	132+27	131+22	1800	2215	Pour/Malfunction in hydraulic from carrier
4-24-72[1]			0700	1630	Strip and Form 105' each/Malfunction in hydraulic from carrier
4-24-72[1]	131+22	130+17	1600	1945	Pour
4-24-72[1]			1600	2430	Clean up ahead of pour
4-25-72[2]	130+17	129+12	1300	1630	Pour and strip/1.5 cubic yards short on pour, rock pocket east side Station 130+17, three cold joints east and west sides Station 130+17 and 130+35
4-25-72[2]			1600	2430	Strip and form 105' each
4-26-72[2]	129+12	128+07	1220		Pour strip and form 105' each/concrete was poured somewhat wet and with ground water caused froms to shift eastward and rise above setting.05'. It was suggested to foreman that more trucks are needed for a pour of this length. Small rock pockets top quarters Station 130+17. Cold joint lower east quarter Station 130+17/Ten Feet of resteel at east springline at Station 128+92

Table 2 (continued)

Date	Concrete Placed		Time		Operation/Problem/Notes
	From	To	From	To	
4-27-72[2]	128+07	127+02	1205	1530	Pour and strip/Small honeycombed spots at bulkhead Station 128+07
4-28-72[2]	127+02	125+97	1300	1720	Pour/After pouring 260 cubic yards drop shaft plugged up, could not move even from top, void in crown approx. ten cubic yards (15'x8') not covering forms
4-29-72[2]	125+97	124+92	1300	1550	Pour and strip/Ordered ten cubic yards extra to fill void left on 4-28-72
5-01-72	124+92	123+87	0815	1335	Pour and strip/At Station 125+8 a stone pocket 4'x1' in the east quarter. At Station 125+45 a stone pocket 6"x5' in the west quarter
5-02-72	123+87	122+82	1000	1255	Pour and strip/V type cold join at invert Station 123+20 and 13' long cold joint at west side of Station 123+30/Resteel at crown from Stations 122+82 to 122+93
5-04-72	122+82	121+77	1640	1940	Pour and strip/Rock seam 14' long lower west quarter at Station 123+87 to 123+73. A 2' dia. rock pocket at Station 123+87 at east springline. Finish grade is .01' down/Resteel in top east quarter from Station 122+47 to 122+82
5-04-72			Day & Afternoon		Strip and form 105' each
5-05-72	121+77	120+72	1300	1630	Pour and strip
5-30-72	120+72	119+67	1355	1715	Pour and strip
5-01-72	119+67	118+62	1700	2130	Pour and strip/One hour and fifteen minute delay due to both drop holes plugging up, went through top doors to start concrete moving/Waterstop at Station 118+62
5-02-72	118+62	117+57	1600	1930	Pour and strip/Small amount of cold joint east and west lower quarters/Waterstop at Station 117+57.

Table 2 (continued)

Date	Concrete Placed		Time		Operation/Problem/Notes
	From	To	From	To	
6-03-72	117+57	116+52	1445	1725	Pour and strip/2'x4' honeycomb at lower quarter east side Station 118+61/Thirty feet of resteel crown at Station 117+42. Waterstop at Station 116+52
6-05-72	116+52	115+47	1425	1825	Pour and strip/2'x1' trace of clay in crown at Station 117+22/Waterstop at Station 115+47
6-05-72			1700	0100	Remove form bolts for 105'
6-06-72[3]	115+47	114+42	1225	1610	Pour and strip/Small rock pockets at start of pour and small cold joints/Waterstop at Station 114+42
6-06-72[3]			1700	2430	Remove form bolts for 105'
6-07-72[3]	114+42	113+37	1205	0304	Pour and strip/Several cold joints both east and west sides approx. 1/2" wide/Waterstop at Station 113+37
6-07-72[3]			1700	2400	Remove form bolts for 105'
6-08-72[3]	113+37	112+32	1230	1540	Pour and strip/Waterstop at Station 112+32
6-08-72[3]			1700	0100	Remove form bolts for 105'
6-09-72[3]	112+32	111+27	1300	1640	Pour and strip/Four small cold joints east and west sides/Thirty five feet of resteel in top from Station 111+47 to 111+82. Waterstop at Station 111+27
6-09-72[3]			1700	0130	Remove form bolts for 105'
6-10-72[3]	111+27	110+22	1430	1650	Pour and strip/Rock pockets east and west springline Station 110+82. Waterstop at Station 110+22
6-12-72	110+22	109+17	1450	1730	Pour and strip/3'x3' honeycomb a east and west side Station 111+27 at springline/Resteel in bottom 1/2 from Station 109+17 to Station 109+52. Waterstop at Station 109+17

Table 2 (concluded)

Date	Concrete Placed From	Concrete Placed To	Time From	Time To	Operation/Problem/Notes
6-14-72	109+17	108+12	1250	1645	Pour and strip//Resteel in top 1/2 from Station 108+12 to 108+47. Steel around entire tunnel from Station 108+47 to 109+02. Waterstop at Station 108+12
6-16-72	108+12	107+07	0925	1300	Pour and strip/Resteel in invert from Station 107+07 to 107+97. Resteel in crown from Station 107+47 to 108+12. Waterstop at Station 107+07
6-17-72	107+07	106+02	1320	1545	Pour and strip/Minor honeycomb east and west side/Resteel in invert from Station 106+02 to 106+67. Waterstop at Station 106+02
6-19-72	106+02	104+97	1415	1745	Pour and strip/Small rock pockets around construction joint at Station 107+07 also clay in crown Station 106+57/Resteel in bottom 1/2 from Station 104+97 to 105+87. Waterstop at Station 104+97
6-20-72	104+97	103+92	1245		Pour and strip/Few small rock pockets throughout pour due to stoney concrete. Surface of concrete is generally good/Resteel at bottom 1/2 from Station 104+27 to 104+82. Two bars in invert entire pour. Waterstop Station 103+92
6-21-72	103+92	103+47	1400	1545	Pour and strip/Minor roughness at invert/Twenty feet of resteel in upper 1/2 at Station 103+77. Waterstop at Station 103+47
6-27-72					Core removed at springline west side at Station 135+73
6-29-72					Two cores removed at Station 123+17 at springline and lower quarter east side
7-05-72					Core removed at crown Station 104+17
7-07-72[2]					Filled void at Station 126+12 and filled power drop hole at Station 123+42

Table 3

Available Information Used in Geotechnical Study

Source	Project	Description of Information
City of Detroit Water and Sewerage Department	Contract PCI-7 PCI-3 and PCI-16	Preconstruction boring logs and soil test data and plans showing boring locations.
	Contract PC-433	Construction boring logs and plans showing boring locations.
	Contract PCI-7	Design calculations. Specifications and as-built drawings. Tunnel Rib, Drawing No. 109-6. Construction Photographs. DMWS construction inspector's shift reports. DMWD Sewer construction daily reports. Project summary reports. "Summary of Contract PCI-7, Inspectors Mining Reports Compared to PCI-3 Boring Logs," by E. Kulick.
	Contract CS-858	Contract drawings for the temporary bypass and pumping shaft. Photographs and video tape of tunnel Distress Areas 1 and 2. Complete set of logs for test borings, piezometer borings, settlement pin borings and dewatering well borings. *Daily Inspection Reports for 1 Feb 80* to 10 Mar 80. Tunnel profiles. Piezometer readings through 23 May 80.
	General	DMWS Soil laboratory test descriptions and legends.
Corps of Engineers Detroit District	PCI-7	Color Photographs of PCI-7 tunnel interior after initial cleanup, Sta 133+75 to Sta 110+15.
	Flood Control Study	Boring logs, plan of boring locations and laboratory test data along Red Run, vicinity of PCI-7.
	Wastewater Management	"Special Report, Geologic Considerations for South East Michigan Wastewater Management," Feb 73 by Andrew Mazola.
U. S. Geological Survey in cooperation with U. S. Army Corps of Engineers	Water Resources	Southeastern Michigan Water Resources Study, Groundwater and Geology, 1975, by F. R. Twenter

Table 4

WES Surface Borings

Boring No.	Location		Sample Type	Purpose
WES-1	116+42	20'E	Split spoon/ Undisturbed	Soil stratification, in situ soil conditions and undisturbed samples from 45 to 67 ft depth in nondistressed area.
WES-2	125+18	22'E	Split spoon	Soil stratification and in situ conditions in nondistressed area.
WES-3	130+20	20'E	Undisturbed	Soil stratification, in situ soil conditions, and undisturbed samples close to south end of Distress Area 1.
WES-4	112+81	18'E	Undisturbed	Soil stratification, in situ soil conditions, and undisturbed samples in Distress Area 3.
WES-5	126+85	25'E	Undisturbed	Soil stratification, in situ soil conditions and undisturbed samples in Distress Area 2.
WES-6	128±50	±20'E	None	Define extent and depth
WES-7	122±80	±20'E	None	of landfill found in WES-2,
WES-8	124±00	±20'E	None	WES-3, and WES-5.
WES-9	114+25	12'E	Split spoon	Define soil strata and in situ conditions adjacent to tunnel
WES-10	113+25	13'W	Split spoon	in Distress Area 3 between depths of 45 to 65 ft.

Table 5. Location of Construction Dewatering Wells 15 Mile Road to Red Run (Source: DMWD Daily Reports, 10-20-70 to 12-14-70)

Station*	Depth of Well ft	Depth to Water During Drilling	Remarks
136+33	-		20 ft west and 20 ft north of 9' 6" tunnel stub
134+05	93	65	Hit rock and water at depth of 65 ft.
133+05	93	65	
132+05	92	68	
131+05	92	70	
130+05	93	68	
129+10	93	65	
128+10	94	68	
126+98	92	72	
125+92	92	67	
124+95	91	61	
124+20	92	67	
123+25	91	63	
122+20	89	69	
121+25	86	69	
120+20	89	61	
119+20	91	57	
118+05	86	57	
116+80	93	55	
115+80	91	57	
114+80	98	-	
113+80	96	72	
112+20	52	52	Hit rock and water at 52 ft.
111+80	99	65	
111+70	50	-	
111+20	105	43	

Table 5 (Continued)

Station*	Depth of Well ft	Depth to Water During Drilling ft	Remarks
110+70	70	-	
110+20	88	43	
109+20	85	-	
108+67	73	68	
108+22	96	40	
107+65	62	50	Hit rock at 62 ft.
107+35	87	73	Hit water and sand at 73.
107+05	91	50	
106+75	65	40	Hit rock at 65
106+20	97	53	
105+80	85	40	

* All wells located 25 ft east of tunnel centerline.

Table 6. Extracts Concerning Dewatering from Detroit Metropolitan Water Department Daily Inspection Reports

Date	Shift	Inspector	Log Entry
10-26-70	Day	E. Karsten	"Tri-State Drilling put down well No. 122 at sta 110+20, depth 88 ft and well No. 123 at sta 111+20, depth 105 ft. No Gas by test with well full of water."
10-27-70	Day	E. Karsten	"Tri-State Drilling put down well No. 124 at sta 106+20, depth 97 ft, no explosive gas by test, but well full of water."
8-4-72	Day	H. Ezekiel	"3 operators and 3 men started removing and pulling deep well casings N. of cofferdam shaft."
8-7, 8, 9, 10, 11, 13, 14-72	Day	H. Ezekiel	"3 operators and 3 men continued to pull deep well casings N. of cofferdam shaft."
8-15, 16-72	Day	H. Ezekiel	"Continued to pull deep well casings S. of 15 Mile Rd, 3 men and 3 operators working."

Table 7

Soil Conditions Noted in DMWS Inspector Daily (Shift) Reports

Date	Station From	Station To	Ground Condition
11-02-71	135+83	135+79	Plastic-like clay with sand throughout
11-08-71	135+47	135+43	Hard, gray plastic-like clay with rocks and sand
11-09-71	135+43	135+35	Firm, gray plastic-like clay with rocks and sand
11-10-71	135+35	135+23	Firm to soft gray plastic-like clay mixed with sand and rocks
11-11-71	135+23	135+07	Firm to soft gray clay with some sand and rocks
11-12-71	135+07	134+95	Firm to soft gray clay with some sand and rocks
11-15-71	134+95	134+83	Firm to soft gray clay with some rocks
11-16-71	134+83	134+63	Plastic-like gray clay with small stones throughout
11-17-71	134+63	134+35	Firm to soft plastic-like gray clay
11-18-71	134+35	134+19	Gravel and sand mixed with firm gray clay, some rocks included
11-19-71	134+19	133+99	Gravel and sand mixed with firm gray clay
11-19-71	133+99	133+83	Gravel and sand mixed with gray clay throughout
11-23-71	133+83	133+63	Sand and gravel with gray clay throughout
11-24-71	133+63	133+35	not recorded
12-06-71	133+35	--	Very hard gray sand mixed with scattered stones and traces of clay
12-07-71	133+35	133+20	Very hard gray sandy clay with stones of various sizes, above lower 1/4 point; gravel vein in very bottom of tunnel
12-07-71	133+20	132+95	Hard gray clay with stones throughout
12-08-71	132+95	132+85	Very hard gray sandy clay with some scattered stones throughout
12-09-71	132+50	132+25	Gray sand and clay with rocks
12-09-71	132+70	132+50	Gray sand mixed with very hard gray sandy seams and scattered stones throughout
12-10-71	132+25	132+05	not recorded
12-10-71	132+05	131+80	not recorded
12-11-71	131+80	131+65	Hard sandy gray clay with scattered stones
12-14-71	131+25	131+05	Hard clay to spring line and sandy gray clay on top
12-14-71	131+55	131+25	Very hard gray clay with stones throughout the bottom and stoney softer clay above spring line
12-15-71	130+90	130+65	Sand and gray clay; dry condition

Table 7 (Continued)

Date	Station From	Station To	Ground Condition
12-15-71	131+05	130+90	Very hard gray sandy clay with scattered stones
12-16-71	130+65	130+35	Hard gray sandy clay with scattered stones of various sizes throughout
12-16-71	130+35	130+25	not recorded
12-17-71	130+25	130+00	Hard gray sandy clay with scattered stones
12-17-71	130+00	129+90	not recorded
12-18-71	129+91	129+56	not recorded
12-20-71	129+56	129+16	Very hard gray clay with hard sandy spots; also many stones of various sizes throughout
12-20-71	129+16	128+76	Rock and gray clay and sand
12-21-71	128+76	128+26	Gray sand mixed with traces of firm gray clay, also pebbles to large stones are prevalent throughout
12-21-71	128+26	127+76	Same as above
12-22-71	127+76	127+56	Clay, very rocky
12-22-71	127+56	127+21	Same as above
12-23-71	127+21	126+96	Very rocky sandy gray clay; large boulders
12-23-71	126+96	126+76	Clay and rocks
12-24-71	126+76	126+61	Very hard gray sandy clay with traces of soft clay in the crown; also scattered stones throughout
12-27-71	126+61	126+21	Same as above
12-28-71	125+81	125+36	Hard clay and sand with rocks; some water started to appear
12-29-71	125+01	124+81	Same as above
12-29-71	124+81	124+41	Clay and large rocks
12-30-71	124+41	124+11	Same as above
12-30-71	124+11	123+76	Large rocks and clay
12-31-71	123+76	123+46	Very hard clay with many stones of various sizes; also some signs of dampness throughout
01-03-72	123+46	123+01	Gray sand and clay with rocks
01-03-72	123+01	122+71	Same as above
01-04-72	122+71	122+61	Same as above
01-04-72	122+31	122+16	Hard, full of rocks
01-05-72	122+16	121+86	Very hard rocky gray sandy clay
01-06-72	121+86	121+51	Firm gray sandy clay with many stones

Table 7 (Continued)

Date	Station From	Station To	Ground Condition
01-06-72	121+51	121+21	Sandy clay and rocks
01-07-72	121+21	120+96	Very hard gray clay with many large rocks
01-07-72	120+96	120+76	Hard clay with stones
01-08-72	120+76	120+61	Very hard and rocky clay, some sand showing on top
01-10-72	120+46	120+06	Very hard gray sandy clay with many stones throughout
01-10-72	120+06	119+91	Rocky
01-11-72	119+91	119+26	Hard gray clay, rocky
01-12-72	119+26	118+81	Very hard gray sandy clay with scattered stones throughout
01-12-72	118+81	118+56	Hard gray clay with boulders
01-13-72	118+56	118+21	Very hard gray stoney clay with some soft clay in top
01-14-72	118+21	117+81	Extremely hard sandy gray clay with a lot of stones of various sizes
01-14-72	117+81	117+46	Hard clay and rocks
01-15-72	117+46	117+06	Very hard gray sandy clay
01-15-72	117+06	116+81	Hard gray clay with rocks; sand on bottom
01-17-72	116+81	116+41	Very firm gray sandy clay with sandy bottom and scattered stones throughout
01-17-72	116+41	116+01	Hard gray clay with boulders
01-18-72	116+01	115+76	Very hard gray rocky clay, with a sandy seam throughout
01-18-72	115+76	115+41	Hard gray clay with large rocks
01-19-72	115+41	115+11	Clay with sand on bottom
01-20-72	115+11	114+86	Very hard gray clay with many large stones
01-20-72	114+86	114+66	Hard gray clay and rocks with traces of sand
01-21-72	114+66	114+36	Very hard gray clay with sand seams in bottom; also many stones of various sizes
01-21-72	114+36	114+06	Hard clay with rocks
01-22-72	114+06	113+07	Firm gray sandy clay with scattered stones throughout
01-22-72	113+71	113+31	Hard clay with rocks; sandy layer in face at springline
01-24-72	113+31	113+11	Hard gray clay with sandy bottom and top; also scattered stones
01-24-72	113+11	112+76	Rocky hard clay with traces of sand and small amount of water

Table 7 (Continued)

Date	Station From	Station To	Ground Condition
01-25-72	112+76	112+46	Firm gray sandy clay with many stones of various sizes
01-25-72	112+46	112+16	Same as above
01-26-72	112+16	111+91	Hard gray clay with scattered stones
01-26-72	111+91	111+56	Hard clay with rocks
01-27-72	111+56	111+31	Firm gray clay with many large stones throughout
01-27-72	111+31	111+01	Hard gray clay but getting softer as indicated by mining machine
01-28-72	111+01	110+71	Hard gray clay with sandy wet bottom and scattered stones
01-28-72	110+71	110+44	Same as above
01-29-72	110+41	110+16	Hard ground and many large rocks
01-29-72	110+16	110+01	Hard clay
01-31-72	110+01	109+71	Very hard gray stoney clay with many large rocks throughout
01-31-72	109+71	109+36	Hard clay and stones throughout
02-01-72	109+36	109+16	Very soft mushy clay with a lot of stones
02-01-72	109+16	108+93	Sand and gravel, little clay
02-02-72	108+93	108+88	Adverse soil mixed with water, sand, silt, and some stones
02-03-72	108+68	108+85	Plastic clay
02-04-72	108+78	108+63	not recorded
02-04-72	108+85	108+78	Sand, silt, water, with some gray clay throughout
02-05-72	108+63	108+58	Sand, gravel, and water, with traces of clay
02-05-72	108+58	108+43	Sand with hard-pan gravel
02-07-72	108+43	108+36	Entire base of mining consists of sand mixed with coarse sand and gravel
02-07-72	108+36	108+31	not recorded
02-08-72	108+31	108+11	not recorded
02-09-72	108+11	107+78.5	Loose sand mixed with sharp sand and gravel; also chunks of clay which appeared to be falling from the above machine, thereby causing voids with signs of settlement clear to the surface
02-09-72	107+78.5	107+65.5	not recorded
05-08-72	107+65.5	107+63	Wet sand mixed with traces of wet clay
05-08-72	107+63	107+58	Very wet mud and sand

Table 7 (Continued)

Date	Station From	Station To	Ground Condition
05-09-72	107+58	107+48	Mixed clay, sand, gravel, and water
05-09-72	107+48	107+38	Soft gray clay, mixed with sand, gravel and water
05-10-72	107+25.5	107+09	Very hard gray clay, with considerable amount of rocks throughout
05-11-72	107+09	106+71.5	Very hard gray clay, with many rocks throughout
05-11-72	107+09	106+86.5	Very hard clay with scattered stones throughout
05-12-72	106+58	106+38	Hard gray clay with scattered stones
05-12-72	106+38	106+18	Hard gray clay with many rocks throughout
05-12-72	106+18	106+03	Hard gray clay with rocks throughout
05-13-72	106+03	105+88	Very hard gray clay with scattered stones and some wet spots within 1/4 of the top and crown
05-13-72	105+88	105+78	Hard gray clay mixed with rocks and sand
05-13-72	105+78	105+68	Dry sand with some clay and stones throughout
05-15-72	105+68	105+58	Hard gray clay mixed with sand and rocks throughout
05-15-72	105+58	105+44.5	Hard gray clay mixed with sand and rocks
05-16-72	105+44.5	105+33	not recorded
05-16-72	105+33	105+12	Very hard gray clay with traces of sand and many rocks
05-16-72	105+12	104+97	Hard gray clay with sand and stones throughout
05-17-72	104+97	104+89.5	Very hard gray clay with scattered stones
05-17-72	104+89.5	104+72	Hard gray clay with many rocks throughout; wet silty sand in crown
05-17-72	104+72	104+60.53	Soft clay and sand in top with hard-pan in bottom; also rocks
05-18-72	104+60.53	104+55.53	Very hard clay approximately 3 ft above springline, with silt and sand above machine
05-18-72	104+55.53	104+28.03	Hard gray clay with considerable amount of rocks throughout; crown shows silty sand and water
05-18-72	104+28.03	104+18.03	Soft clay and sand with stones throughout
05-19-72	104+18	104+15.5	Firm gray clay
05-23-72	104+15.5	104+03	Very soft, mud-like clay mixed with silt and water
05-23-72	104+03	103+93	Very soft mud and sand mixed with rocks throughout

Table 8
Summary of Grouting Operations
15 Mile Road/Edison Corridor Tunnel Failure Study

Grout Hole	Location	Date Installed	Date Grouted	Quantity of Grout
2	128+66 - 10' E	2/2/80	2/7/80	0 cu ft
3	128+67 - 12' W	2/5/80	2/7/80	0 cu ft
4	131+83 - 12' W	2/3/80	2/4/80	110 cu ft
5	131+83 - 9' E	2/1/80	2/2&3/80	570 cu ft
6	132+23 - 8' E	2/5/80	2/6/80	150 cu ft
8A	132+40 - 9' E	2/5/80	2/6/80	0 cu ft
10	131+66 - 11' E	2/4/80	2/5/80	200 cu ft
11	131+51 - 11' W	2/4/80	2/6/80	50 cu ft
12	131+72 - 11' W	2/4/80	2/4&5/80	313 cu ft
15	128+46 - 9' E	2/6/80	2/7/80	0 cu t
16	128+52 - 12' W	2/6/80	2/7/80	0 cu ft
17	128+26 - 9' E	2/6/80	2/7/80	27 cu ft

Table 9

Summary of Dates of TB Borings Pressure Grouting and First Settlement Pin Reading

Station	West Side of Tunnel			Tunnel Crown Settlement Pins		East Side of Tunnel			
	TB Boring	Date Completed	Date/ Cu. Ft. Grouted	Pin No.	First Reading	TB Boring	Date Completed	Date/ Cu. Ft. Grouted	Tunnel Segment
133+75									
						8A	2-5-80	2-6/0	
	26	2-8-80	NG						
						8B	2-5-80	NG	
				7	2-5-80				Distressed
						6	2-5-80	2-6/150	
				6	2-5-80				
	13	2-5-80	NG			9	2-3-80	NG	Area
				5	2-5-80				
	4	2-3-80	2-4/110			5	2-1-80	2-2, 2-3/570	
									No.
				9	2-6-80				
	12	2-4-80	2-4, 2-5/313			10	2-4-80	2-5/200	1
				8	2-5-80				
	11	2-4-80	2-6/50			24	2-7-80	NG	
130+75									
129+10									
	23	2-7-80	NG	4	2-5-80	22	2-7-80	NG	North
	20	2-7-80	NG			19	2-7-80	NG	of
				2	2-5-80				
	3	2-5-80	2-7/0			2	2-2-80	2-7/0	Distressed
				1	2-5-80				
	16	2-6-80	2-7/0						Area
						15	2-6-80	2-7/0	
	18	2-6-80	NG	3	2-5-80				No.
						17	2-6-80	2-7/27	
128+25									2

NG = Not Grouted

Table 10

Summary of Water Leakage from Mineral Buildup at Joints, Sta 123+00 to 115+00, Nondistressed Area

Station	Type of Joint	Tunnel Side	Type of Mineral and/or Amount of Staining or Crust
122+35	Circumferential crack	--	Heavy
122+07	Cold	--	
121±75	Construction	East and West	Heavy iron crusts
120±70	Construction	East and West	Heavy iron crusts
119±70	Construction	East and West	Heavy iron crusts
119+20 to 119+30	Cold	East and West	Numerous zones, heavy
119+00	Cold	East	Slight stain
118±62	Construction	East and West	Slight stain
117+07	Unknown	East and West	Moderate
116±50	Construction	East and West	Heavy
116+10	Unknown	East and West	Heavy
116+00 to 115+95	Cold	--	Moderate
115+45	Construction	--	Slight
115+35	Crack	--	Moderate (had been epoxied)

Table 11. Summary of Soil Conditions Behind Tunnel Wall

Station	Location: Tunnel Side	Location: Clock Hr North	Concrete Thickness† ft	Lagging Thickness ft	Pipe Sample ft	Soil Description and Remarks	Cone Penetration Data (30 deg cone, 0.5 in.^{2}): Cone Reading psi	Cone Penetration Data: Remarks
132+70	E Wall	4:00	1.3	0.3	1.6 - 3.0	Lean silty clay and sand, gray, dense, 1-in. R-bar at 0.7 ft	-	Not tested
132+70	Invert	6:00	1.3	0.15±	2.5 - 3.0*	Sandy silt, gray	-	Not tested
132+70	W Wall	8:00	1.3	0.30	1.6 - 3.4	Silty clay, gray		
132+60	E Wall	4:00	1.35	-	1.65 - 2.05	Sand, fine, gray	-	Not tested
132+60	Invert	6:00	1.3	0.2±	2.5 - 3.0*	Sandy silt, gray	-	Not tested
132+60	W Wall	8:00	1.55	0.30	1.85 - 2.3	Silty sand, fine, clay lenses, gray	-	Not tested
132+50	E Wall	4:00	1.25	0.30	1.6 - 2.0	Silty sand, same as above	-	Not tested
132+50	Invert	6:00	1.35	0.15±	2.5 - 3.1*	Sandy silt, gray, dense, clay lenses	-	Not tested
132+50	W Wall	8:00	1.25	0.3	1.6 - 2.0	Silty sand, clay lenses	-	Not tested
132+42	E Wall	4:00	1.4	0.3	1.7 - 2.1	Silty clay, small gravel	-	Not tested
132+42	Invert	6:00	1.5	0.15±	2.2 - 2.8*	Sandy silt, gray	-	Not tested
132+42	W Wall	8:00	1.4	0.3	1.7 - 2.1	Silty sand, fine, gray	-	Not tested
132+36	E Wall	3:30	1.5	0.3	1.9 - 2.4	Silty sand, fine, gray, frozen, One 1/2-in. piece gravel	-	Not tested
132+35	E Wall	4:00	1.35	0.3	1.8 - 2.2	Silty clay and silt, gray, frozen	-	Not tested
132+35	Invert	6:00	1.4	0.15±	2.5 - 2.8*	Sandy silt, gray, dense, moderate cohesion	-	Not tested
132+35	W Wall	8:00	1.35	0.30	1.7 - 2.0	Silty sand, fine, gray, frozen	-	Not tested
132+22	E Wall	3:00	1.4	0.3	1.7 - 2.0	Silty sand, fine with clay lenses, gray, semi-frozen	-	Not tested
132+22	E Wall	4:00	1.45	0.30	1.8 - 2.2	Silty sand, fine, gray, semi-frozen	-	Not tested
132+22	Invert	6:00	1.45	0.15±	1.8 - 2.4*	Silty sand, fine, gray, dense, moderate cohesion	-	Not tested
132+22	W Wall	8:00	1.45	0.35	1.8 - 2.2	Sandy silt, gray semi-frozen	-	Not tested
132+22	W Wall	9:00	1.55	0.30	1.85 - 2.3	Silty clay and silt, gray semi-frozen	-	Not tested

(Continued)

Table 11 (Continued)

Station	Location: Tunnel Side	Location: Clock Hr North	Concrete Thickness† ft	Lagging Thickness ft	Pipe Sample ft	Soil Description and Remarks	Cone Penetration Data (30 deg cone, 0.5 in.^{2}): Cone Reading psi	Cone Penetration Data: Remarks
132+12	E Wall	3:00	1.3	0.3	1.6 - 2.0	Silty clay and silt, gray, frozen, one piece 1/2-in. gravel	-	Not tested
132+12	E Wall	4:00	1.3	0.3	1.6 - 2.0	Silty clay and silt, gray, frozen	-	Not tested
132+12	Invert	6:00	1.4	0.15±	1.7 - 2.0	Sandy silt, gray, dense, moderate cohesion	-	Not tested
132+12	W Wall	8:00	1.35	0.35	1.7 - 2.1	Silty sand, fine, gray, frozen	-	Not tested
132+12	W Wall	9:00	1.5	0.3	1.9 - 2.2	Silty clay, gray, frozen, 0.5 percent fine gravel	-	Not tested
130+25	E Wall	3:00	1.35	0.3	1.7 - 2.3	Clay, gray, firm, moist, CL, 5 percent fine gravel	170**	
130+25	W Wall	9:00	1.35	0.3	1.6 - 2.7	Sand, very fine, gray, moderate cohesion, clay seam, gray, dense, CL	170**	
130+10	E Wall	3:00	1.3	0.3	1.6 - 2.4	Clay, gray, firm, moist, CL, 5 percent grvel	170**	
129+12	E Wall	3:45	-	-	-	Silty sand, moderate cohesion, dry	220**	
129+12	W Wall	8:00	1.4	0.3	1.7 - 2.6	Silty sand, no cohesion, dry	240**	
128+52	E Wall	3:00	1.4	0.3	1.7 - 2.6	Silty sand and sand	140**	
128+52	W Wall	9:00	1.5	0.3	1.8 - 2.9	Silty sand, moderate cohesion, dry	170**	
128+39	E Wall	3:00	1.3	0.3	2.1 - 2.6	Silty sand, moderate cohesion, dry	300+	Pushed to cone shoulder (1.5-in. penetration)
128+07	E Wall	3:00	1.3	0.3	1.6 - 3.3	Silty sand, very fine, gray, moderate cohesion, moist	180	1.5-in. penetration
128+07	E Wall	4:00	1.6	0.3	1.9 - 3.0	Clay contact at lagging into silty sand, moderate cohesion, dry	170	6-in. penetration
128+07	W Wall	8:00	1.6	0.3	1.9 - 2.8	Clay contact at lagging into silty sand, moderate cohesion dry	300	1.5-in. penetration

(Continued)

Table 11 (Continued)

	Location						Cone Penetration Data (30 deg cone, 0.5 in.^{2})	
Station	Tunnel Side	Clock Hr North	Concrete Thickness† ft	Lagging Thickness ft	Pipe Sample ft	Soil Description and Remarks	Cone Reading psi	Remarks
127+02	E Wall	3:30	1.3	0.3	1.9 - 3.0	Clay, gray, soft, moist, CL	220	5.4-in. penetration
127+02	W Wall	8:30	1.4	0.3	1.7 - 2.8	Clay, same as above	140	1.5-in. penetration
126+12	E Wall	3:00	1.7	0.3	2.2 - 2.8	Sandy clay, very fine, sand with approximately 30 percent clay, gray, moist, dense	170**	
126+12	E Wall	3:30	1.8	0.3	2.3 - 3.2	Silty sand, fine, gray, moderate cohesion, moist	180**	
126+12	W Wall	8:30	1.3	0.3	1.7 -	Silty sand, fine, gray, moderate cohesion, moist	240**	
125+97	E Wall	3:30	1.5	0.3	2.0 - 3.0	Clay, gray, dense, CL, wet at lagging, dryer with depth	220**	
125+97	W Wall	8:30	1.5	0.3	1.9 - 2.4	Silty sand, fine, gray with clay, moist, dense	250**	
124+92	E Wall	3:30	1.7	0.3	2.1 - 3.6	Silty sand (same as above)	220**	1.5-in. penetration
124+92	W Wall	8:30	1.5	0.3	1.9 -	Silty sand (same as above)	200**	
123+87	E Wall	4:00	1.5	0.3	1.9 -	Sandy silt, very soft, moderate cohesion, gray, CL, moist	60 - 80	18-in. penetrtion
123+87	W Wall	8:00	1.6	0.3	2.1 - 2.8	Silty sand, fine, gray, moist	210	5.4-in. penetration
122+82	E Wall	4:00	1.5	0.3	1.8 - 3.3	Sandy clay, gray, moist, CL (very soft at the back of lagging)	210**	
122+82	W Wall	8:00	1.7	0.3	2.3 - 3.5	Sandy clay, gray, dense, CL	230	1/2-in. penetration
121+77	E Wall	4:00	1.4	0.3	1.7 - 3.0	Silty sand, fine, gray	220**	Previously sampled hole, dry
121+77	W Wall	8:00	1.55	0.3	1.85 - 3.0	Silty sand, fine, gray	170**	Previously sampled hole, dry
120+20	E Wall	4:00	1.5	0.3	1.8 - 3.0	Silty sand, fine, gray	140**	Previously sampled hole, wet
120+20	W Wall	8:00	1.5	0.3	1.8 - 3.0	Silty sand, fine, gray	165**	Previously sampled hole, wet

(Continued)

Table 11 (Concluded)

Station	Location: Tunnel Side	Location: Clock Hr North	Concrete Thickness† ft	Lagging Thickness ft	Pipe Sample ft	Soil Description and Remarks	Cone Penetration Data (30 deg cone, 0.5 in.^{2}): Cone Reading psi	Cone Penetration Data: Remarks
119+67	E Wall	4:00	1.6	0.3	1.9 - 2.5	Silty sand, fine, gray	160**	Previously sampled hole, wet
119+67	W Wall	8:00	1.4	0.3	1.7 - 2.5	Silty sand, fine, gray, wet, no cohesion, dense	190**	Previously sampled hole, wet
118+00	E Wall	4:00	1.6	0.3	1.9 - 3.0	Silty sand same as above	185**	Previously sampled hole, filled in and wet
118+00	W Wall	8:00	1.6	0.3	1.9 - 2.8	Silty sand, same as above, Rib at 1.4 ft	140**	Previously sampled hole, filled in and wet
116+50	E Wall	3:45	1.5	0.3	1.8 - 2.5	Sandy clay, clay (CL) is firm, moist with 5 percent fine gravel	160**	Previously sampled hole, water at 0.8 ft depth
116+50	W Wall	8:15	1.6	0.3	1.9 - 2.6	Sandy clay, same as above	150**	Pushed in previously sampled hole, wet
115+45	E Wall	3:00	1.5	0.3	1.8 - 2.5	Silty sand, fine, gray, dense, little cohesion, 5 percent fine gravel, dry, dewatered	-	Not tested
115+45	W Wall	9:00	1.65	0.3	1.95 - 2.60	Silty sand, fine, gray, dense, dry	-	Not tested
113+25	E Wall	*4:00	1.4	0.3	1.9 - 2.8	Silty sand to sandy silt, gray moist, moderate cohesion	180**	
113+25	W Wall	8:00	1.4	0.3	1.9 - 2.5	Silty sand to sandy silt, gray, moist, moderate cohesion	140**	

† Holes not always perpendicular to wall and concrete thickness may not be minimum value
* *Used 2-in.-diameter hand auger*
** Maximum force possible by hand pushing, no apparent penetration

Table 12

Construction Joint Conditions, Distressed Area 3

Station	Remarks
114+42	Sandy silt inclusion behind 4-in. loose core piece, waterstop, piece of wet, partially decayed wood, lower east quarter; waterstop and wood higher up in joint on west side
113+37	Waterstop and piece of wood in keyway, east and west sides of tunnel
112+32	Piece of waterstop visible where fractured concrete had fallen out of crown
111+27	Waterstop and wood in keyway, west side, lower quarter, joint leaking water, "epoxy" written on wall

Table

BORING NO.	SAM. NO.	Averg. DEPTH OF SAMPLE	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10}	ATTERBERG LIMITS LL	PL	SPECIFIC GRAVITY G	NAT WATER CONT %	NATURAL DRY DENSITY LBS/CU FT	COMPACTION D OPT WATER %	MAXIM DRY DE LBS/CU
WES-1	32	53.8	Sandy Clay CL	3	15	82		27	15	2.74				
	34	60.3	Silty SandSP-SM	0	93	7						131.4*		
	38	64.9	Sand SP	0	98	2						107.6*		
WES-3	6	37.1	Silty Clay CH								20.6	110.6		
	9	41.6	Silty Clay CL					22	11	2.76	14.6	124.7		
	12	46.6	Silty Clay CL					24	13	2.70	18.8	108.3		
	16	52.4	Grav.Silty Sand											
			SC-SM	14	48	38		20	14		11.4	127.7		
	18	55.3	Sandy Clay CL					25	13	2.74	12.4	125.0		
	19	56.0	Silty Clay CL					26	14	2.74	16.7	117.7		
	19	56.0	" " "								11.9	125.9		
	23	62.0	Silty Clay CL	0	3	97					14.9	123.0		
	24	63.0	SiltySandSP-SM								15.2	118.5		
	31	75.2	Sandy Silt ML	0	50	50						117.0*		
WES-4	4	4.8	Silty Clay CL					30	18		26.1	101.3		
	8	10.7	Silty Clay CL					32	18	2.74	29.3	95.7		
	12	21.4	Silty Clay CL					29	16	2.76	14.0	121.4		
	13	25.8	Silty Clay CL					31	16	2.75	15.5	122.9		
	17	36.6	Silty Clay CL					24	14	2.75	12.4	131.3		
	20	45.2	Clayey Sand SC							2.72		129.6*		
	22	47.8	Sand w/Grav.SP	6	93	1		14	9		9.2	139.5		
	24	50.6	Sandy Clay CL	4	13	83		28	12	2.74	16.5	126.2		
	24	50.6	" " "								13.9	121.2		
	25	52.5	Sandy Clay CL	0	18	82		24	12	2.75**	10.5	132.6		
	27	55.5	Silty Clay CL					24	13	2.73	12.0	142.2		
	30	64.5	Silt ML	0	3	97					18.3	112.6		
	31	66.6	Sandy Silt ML	0	38	62					19.2	114.0		
	32	69.5	Sandy Silt ML	0	13	87						120.6*		
WES-9	3	51.8	Silty ClayCL-ML	0	24	76		16	10					
	4	60.8	Silt ML	0	16	84		17	14					
	6	64.3	Silty Sand SM	0	80	20		NP						
WES-10	2	48.8	Clayey Silt ML	0	34	66		13	10					
	5	57.8	Silty ClayCL-ML	0	18	82		17	12					
	7	63.8	Silty ClayCL-ML	0	22	78		16	12					

Table 13. **TEST DATA SUMMARY**

PROJECT 15 Mile Road/Edison Corridor Tunnel Failure Study

COMPACTION DATA		SHEAR DATA												PERMEABILITY		
OPT WATER %	MAXIMUM DRY DENSITY LBS CU FT	INITIAL e	DRY DENSITY LBS/CU FT	W_1 %	W_F %	S_1 %	TYPE TEST	SPECIMEN SIZE INCHES	TEST	σ_m T/SQ FT	σ_1 T/SQ FT	c T/SQ FT	ϕ DEGREES	e	K FT/MIN.	T/
		0.381	124.7	14.6	14.2	100	DS	3 x 3				0.15	26.5			
																3.
			125.9	11.9	12.7	100	K_o $\bar{R}$	2.8 x 6.5				0	33.9			
			118.5	15.2	15.4	100	K_o $\bar{R}$	2.8 x 6.4				0	35.9			
		0.787	95.7	24.3	32.2	100	DS	3 x 3				0.10	24.0			
																1.
		0.396	122.9	15.5	17.4	100	DS	3 x 3				0.08	30.5			
		0.348	126.2	16.5	15.8	100	DS	3 x 3				0	28.5			
			121.2	13.9	13.4	100	K_o $\bar{R}$	2.9 x 6.4				0.06	31.7			
		0.294	132.6	10.5	12.5	98.5	DS	3 x 3				0	33.5			
			112.6	18.3	17.5	100	K_o $\bar{R}$	2.8 x 6.3				0	36.4			

* - Wet density
** - Estimate

T – TRIAXIAL COMPRESSION
UC – UNCONFINED COMPRESSION

DS – DIRECT SHEAR
Q – UNCONSOLIDAT

2

SHEET ____ OF ____

[illegible] k FT/MIN	CONSOLIDATION DATA P T/SQ FT	P T/SQ FT	C	[illegible]	REMARKS
					Piping Test
					Piping Test
					Total Sample
					Total Sample
					Total Sample
	3.32	[illegible]			
					Total Sample
					T
					Total Sample
					T
					Piping Test
					Total Sample
	1.[illegible]	5.3			
					Total Sample
					Piping Test
					Total Sample
					T
					Total Sample
					T
					Total Sample
					Piping Test
					Jar Sample
					Jar Sample
					Jar Sample
					Jar Sample
					Jar Sample
					Jar Sample

[illegible]ECT SHEAR
[illegible]CONSOLIDATED UNDRAINED

S – CONSOLIDATED DRAINED
R – CONSOLIDATED UNDRAINED

3

Table 14

List of Samples and Results of Special Piping Tests

Test No.	Sample Number	Sample Depth ft	Sample Diam in.	Saturation Time hr	Saturation Pressure psi	Piping Water Pressure, psi Initial	Piping Water Pressure, psi Final	Cumulative Piping Time min	Classification	Material Density pcf	Material Dry Wt g	Material Wt Piped g	Material % Piped
1	WES-1-38-1*	63.9-64.9	3	20	1/2	3/4	5	7	Sand (SP) Gray	107.56	2100	67	3.19
2	WES-1-38-2*	64.9-65.9	3	90	1/2	2	3	53	Sand (SP) Gray	--	--	--	--
3	WES-4-20*	44.7-45.8	5	19.33	1/2	2	2	1	Clayey Fine Sand (SC)	129.60	5808	--	--
4	WES-4-20†	44.7-45.8	5	1.25	1/4	2	8	70	Clayey Fine Sand (SC)	129.60	"	--	--
5	WES-4-32	69.0-70.0	3	19	3-1/4	3	5	55	Sandy Silt (ML)	120.60	2336	370	15.84
6	WES-1-34	59.5-61.1	5	41	2-1/2	3	8	55	Soft Silty Sand (SM)	131.36	8830	248	2.81
7	WES-3-31	74.4-76.0	5	98	1/2	5	9	35	Sandy Silt (ML)	117.00	8921	94	1.05
8	WES-5-10	65.0-66.0	5	114	1	1-3/4	8	55	Clay and Silt (CL-ML)	--	5328	21	0.39

* Slot cut longitudinally.

† Longitudinal slot plugged and circular slots cut in a second test.

-- No material piped out of slot.

Table 15

Summary of $K_o\overline{R}$ Triaxial Compression Tests

Boring No.	Sample No.	Depth, ft	Soil Class.	Water Content, %	Wet Unit Weight, pcf	σ'_3 End of Consol., tsf	K_o	E_i Modulus, psi	c Cohesion, tsf	ϕ' Degrees
WES-3	19	55.6/56.4	CL	11.9	141.4	0.83	0.40	3470	0	33.9
WES-3	24	62.6/63.4	SP&ML	15.4	135.9	1.03	0.45	4550	0	35.9
WES-4	24	50/51.1	CL	13.9	137.4	1.30	0.42	1820	0.06	31.7
WES-4	30	64/65	ML	18.3	132.4	1.50	0.42	3900	0	36.4

Table 16

Modulus and Shear Strength Data, Midland Nuclear Power Station, Michigan
(from Midland Final Safety Analysis Report, Vol 4, Revision 21, 5/79)

Depth Ft	Soil Description	w %	γ_d/γ_m pcf	G_s	Consol σ_3 tsf	Strain Rate %/Hr	E_i(1) psi	c' tsf	ϕ' deg
17.0	Gray sandy silt	8.1	136/147	--	0.5	0.69	3470		
--	and silty clay	13.4	125/142	--	1.0	0.16	1736		
14.5	with some sand	7.3	139/149	2.62	2.0	0.77	6940		
--	and gravel	9.6	135/148	2.71	0.5	0.74	2080	0.3	32
17.5	same as above	8.9	136/148	--	0.5	0.42	2080		
--		10.7	131/145	2.70	1.0	0.17	2290		
44.5		7.3	140/150	--	2.0	0.20	4650	0.3	35

Table 17

Summary of Concrete Information from Various Sources

Date of Placing	Location	Mixture Proportions: Mixture Report No.	Mixture Proportions: Coarse Aggregate lb	Mixture Proportions: Fine Aggregate lb	Mixture Proportions: Type I Cement lb	Mixture Proportions: Water gal	Mixture Proportions: Admixture oz	Slump in.	Time of Placement hr:min	Approx Rate of Placing, cu yd/hr	Cylinder Test Report No.	Compressive Strength: Age, days	Compressive Strength: psi	Remarks from Inspector Reports
17 Apr 72	133+32-132+27	614	1850	1540	517	23	3.2	6	5:25	50	627	14	3450	Small honeycombs on springing line east and west, Sta 133+32
21 Apr 72	132+27-131+22	NA*	--	--	--	--	--	5-1/2	4:15	64	627	7	3150	(Inspectors report shows concrete was placed on 20 Apr 72 whereas the cylinder test report shows it being placed on 21 Apr 72)
24 Apr 72	131+22-130+17	NA	--	--	--	--	--	5-1/2	3:45	71	632	7	2210	Rock pocket, east side Sta 130+19 three cold joints east and west-sides, Sta 130+17 to 130+35
25 Apr 72	130+17-129+12	627	1850	1540	517	21	3.2	6	3:30	77	632	7	3380	Small rock pockets in top 1/4 Sta 130+17; cold joint in lower 1/4 east Sta 130+10
26 Apr 72	129+12-128+07	NA	--	--	--	--	--	5	**	**	644	7	3890	Small honeycombed spots at bulkhead, Sta 128+07
27 Apr 72	128+07-127+02	632	1850	1540	517	23	3.2	5-1/2	3:25	78	644	7	4000	
28 Apr 72	127+02-125+97	NA	--	--	--	--	--	4-1/2	**	**	644	7	3910	Section appeared okay. (No inspectors report for the placement)
29 Apr 72	125+97-124+92	NA	--	--	--	--	--	6	2:50	99	644	9	4240	Stone pockets: 1 ft by 4 ft, east 1/4, Sta 125+80 and 6 in. by 5 ft, west 1/4, Sta 125+45
1 May 72	124+92-123+87	644	1850	1540	517	22	3.2	5	5:20	50	659	14	3930	"V" type cold joint in invert, Sta 124+30; 13 ft long cold joint westside Sta 124+30
2 May 72	123+87-122+82	NA	--	--	--	--	--	5-1/2	2:45	98	659	7	4070	Rock seam, 14 ft long, lower 1/4 west, Sta 123+87 to 123+73; 2 ft diam pocket, Sta 123+87, east springline
4 May 72	122+82-121+77	NA	--	--	--	--	--	6	3:00	90	659	7	2820	
5 May 72	121+77-120+72	NA	--	--	--	--	--	5	3:30	77	659	7	3450	
9 May 72	--	659	1850	1540	517	22	3.2	--	--	--	--	--	--	(No placement recorded on 9 May 72)

(Continued)

* Not available.
** Not given.

Table 17 (Continued)

Date of Placing	Location	Mixture Proportions: Mixture Report No.	Mixture Proportions: Coarse Aggregate lb	Mixture Proportions: Fine Aggregate lb	Mixture Proportions: Type I Cement lb	Mixture Proportions: Water gal	Mixture Proportions: Admixture oz	Slump in.	Time of Placement hr:min	Approx Rate of Placing, cu yd/hr	Cylinder Test Report No.	Compressive Strength: Age, days	Compressive Strength: psi	Remarks from Inspector Reports
30 May 72	120+22-119+67	NA	--	--	--	--	--	5	3:20	80	710	7	3630	
1 Jun 72	119+67-118+62	NA	--	--	--	--	--	4-1/2	4:30	60	710	7	3640	Small amount of cold joint, east and west, L/Q
2 Jun 72	118+62-117+57	NA	--	--	--	--	--	5	3:30	76	710	7	3450	2 ft by 4 ft honeycomb L/Q, east side, Sta 118+61
3 Jun 72	117+57-116+52	NA	--	--	--	--	--	5	2:40	100	710	13	3890	2 ft by 1 ft trace of clay in crown at Sta 117+22
5 Jun 72	116+52-115+47	710	1850	1540	517	23	3.2	5-1/2	4:00	67	720	7	3640	Small rock pockets at start of pour and small cold joints
6 Jun 72	115+47-114+42	NA	--	--	--	--	--	5-1/2	3:45	70	720	7	3060	Several cold joints both east and west side, approx 1/2 in. wide
7 Jun 72	114+42-113+37	NA	--	--	--	--	--	6	3:00	87	720	7	2920	
8 Jun 72	113+37-112+32	NA	--	--	--	--	--	5	3:10	**	720	7	3360	4 small cold joints, east and west sides
9 Jun 72	112+32-111+27	NA	--	--	--	--	--	6	3:40	72	720	7	3420	Rock pockets, east and west, springing live, Sta 112+32
10 Jun 72	111+27-110+22	NA	--	--	--	--	--	6	2:20	112	720	13	4170	3 ft by 3 ft honeycomb, east and west sides, Sta 111+27 at springing line

** Not given.

Table 18

Summary of Schmidt Hammer Rebound Number Measurements on Drilled Cores

Station	Tunnel Side	Clock hr North	Rebound Number: Tunnel Location	Rebound Number: Core End	Rebound Number: Core Side	Compressive Strength of Core psi
133+13	Crown	12:00	--	48	41	7260
	Invert	6:00	--	--	34	--
132+50	Invert	6:00	33	29	37	7510
	W. Wall	9:00	33	41	35	--
	Crown	12:00	26	41	33	6690
131+84	Crown	Not known	--	33	31	6620
	Crown	Not known	--	36	37	7220
131+60	Crown	Not known	--	34	34	7790
	Crown	Not known	--	36	32	7590
127+50	Invert	6:00	27	38	33	7870
	W. Wall	9:00	42	36	35	7640
	Crown	12:00	36	34	25	6690
119+15	E. Wall	2:00	38	42	37	--
	E. Wall	4:00	38	38	33	7060
118+35	E. Wall	1:00	25	45	40	6250
117+70	E. Wall	5:00	38	42	37	6370
117+25	E. Wall	3:00	41	46	34	6280
	Invert	6:00	35	34	39	6440
	Crown	12:00	48	41	39	7060
115+47	W. Wall	8:00	--	--	--	--
114+70	Invert	6:00	31	--	--	--
	Invert	6:30	31	30	43	4440
	W. Wall	9:00	42	44	34	6070
	Crown	12:00	40	39	34	5480
113+45	E. Wall	3:00	34	39	35	6740
	Invert	6:00	28	28	34	6400
	W. Wall	7:00	37	--	--	--
	Crown	12:00	40	37	34	5260
112+20	E. Wall	3:00	37	38	38	5560
	Crown	12:00	46	42	38	--
112+18.5	Invert	6:00	31	31	41	6190

Table 19

Summary of Schmidt Hammer Rebound Number Measurements on In-Place Concrete

105-ft Placement Station Numbers	Location of Readings	Average Rebound Number for Specific Wall Locations			
		Crown	East Wall	Invert	West Wall
134+37 - 133+32	133+50	42	39	32	44
133+32 - 132+27	132+50	26	32	33	33
132+27 - 131+22	--	--	--	--	--
131+22 - 130+17	131+00	45	43	22	39
130+17 - 129+12	130+00	38	34	19	45
129+12 - 128+07	128+50	38	42	19	38
128+07 - 127+02	127+50	36	43	27	42
127+02 - 125+97	126+25	35	43	16	42
125+97 - 124+92	--	--	--	--	--
124+92 - 123+87	124+00	37	39	41	39
123+87 - 122+82	--	--	--	--	--
122+82 - 121+77	--	--	--	--	--
121+77 - 120+72	121+00	33	37	26	35
120+72 - 119+67	--	--	--	--	--
119+67 - 118+62	119+15	43	38	25	43
118+62 - 117+57	118+55	41	35	25	36
	118+35	32	25	25	41
	118+00	41	39	23	40
	117+70	41	38	23	40
117+57 - 116+52	117+25	48	41	35	42
116+52 - 115+47	--	--	--	--	--
115+47 - 114+42	115+00	46	33	26	38
	114+75	42	40	21	43
	114+70	40	43	31	42
	114+50	41	35	26	32
114+42 - 113+37	114+25	46	35	37	42
	114+00	42	41	39	42
	113+75	38	38	33	37
	113+50	42	36	32	41
	113+45	40	34	28	37

Table 19 (concluded)

105-ft Placement Station Numbers	Location of Readings	Average Rebound Number for Specific Wall Locations			
		Crown	East Wall	Invert	West Wall
113+37 - 112+32	113+25	42	44	20	40
	113+00	39	41	20	40
	112+75	37	40	39	37
	112+50	43	42	24	39
112+32 - 111+27	112+20	46	37	31	35
	112+15	43	32	32	37
	112+00	42	26	27	40
	111+75	42	39	33	34
	111+50	43	38	29	41
111+27 - 110+22	111+25	42	25	18	37
	111+00	38	32	26	41

Table 20

Summary of Concrete Thicknesses from Drilled Core Measurements

Station	Location: Tunnel Side	Location: Clock hr North	Concrete Thickness ft
133+13	Invert	6:00	1.2
	E. Wall	3:00	1.2
	Crown	12:00	2.0
132+50	Invert	6:00	1.35
	W. Wall	9:00	1.5
	Crown	12:00	1.5
131+84	Crown	Not known	1.65
	Crown	Not known	1.65
131+60	Large Piece	--	1.1*
(Approx)	Assumed to be from the Crown	--	1.1
127+50	Invert	6:00	1.35
	W. Wall	9:00	1.4
	Crown	12:00	1.7
119+15	E. Wall	4:00	1.45
	E. Wall	2:00	1.35
118+35	E. Wall	1:00	1.35
117+70	E. Wall	5:00	1.6
117+25	Invert	6:00	1.9
	E. Wall	3:00	1.45
	Crown	12:00	1.1
115+47	W. Wall	8:00	1.5
114+70	Invert	6:00	1.6
	Invert	6:30	1.6
	W. Wall	9:00	1.3
	Crown	12:00	1.65
113+45	Invert	6:00	1.65
	W. Wall	7:00	1.6
	E. Wall	3:00	1.6
	Crown	12:00	1.15
112+20	E. Wall	3:00	1.6
	Crown	12:00	1.7
112+18.5	Invert	6:00	1.35

*Not actual liner thickness

Table 21

Summary of Physical Test Data from Drilled Concrete Cores

Station	Tunnel Side	Clock hr North	Compressive Strength psi	Modulus of Elasticity 10^6 psi	Poisson's Ratio
133+13	E. Wall	3:00	--	--	--
	Invert	6:00	--	--	--
	Crown	12:00	7260	--	--
132+50	Invert	6:00	7510	5.05	0.20
	W. Wall	9:00	--	--	--
	Crown	12:00	6690	4.80	0.19
131+84	Crown	Not known		--	--
	Crown	Not known		--	--
131+60	Crown	--	7790	5.90	0.29
	(Assumed)	--	7590	5.05	0.20
127+50	Invert	6:00	7870	4.05	0.15
	W. Wall	9:00	7640	6.50	0.24
	Crown	12:00	6690	5.15	0.28
119+15	E. Wall	2:00	--	--	--
	E. Wall	4:00	7060	--	--
118+35	E. Wall	1:00	6250	4.80	0.20
117+70	E. Wall	5:00	6370	4.75	0.20
117+25	E. Wall	3:00	6280	4.75	0.16
	Invert	6:00	7060	4.75	0.20
	Crown	12:00	6440	4.85	0.19
115+47	W. Wall	8:00	--	--	--
114+70	Invert	6:00	--	--	--
	Invert	6:30	4440	--	--
	W. Wall	9:00	6070	--	--
	Crown	12:00	5480	--	--
113+45	E. Wall	3:00	6740	5.00	0.22
	Invert	6:00	6400	4.70	0.16
	W. Wall	7:00	--	--	--
	Crown	12:00	5260	4.40	0.19
112+20	E. Wall	3:00	5560	--	--
	Crown	12:00	--	--	--
112+18.5	Invert	6:00	6190	--	--

APPENDIX A

BORING LOGS

BORING LOG
FIELD DATA

Project DETROIT SEWER Site ______ Date ______
Location STA 116 + 42 , 20'E ℄ SEWER Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	BLOWS		CLASSIFICATION AND REMARKS
		0.0						Fishtail			Cleanout Clay fill
			4.0								
		4.0		5.8	6.3				1		10 blows/ft. Brown soft silt fill
				6.3	6.8				4		grading to brown-gray soft silty clay
1	3/8/81		7.2	6.8	7.3	7.0	7.3		6		CL at 7.2' Oxidized (soil zone)
											Varved clay (lake deposit)
				7.3	8.5			Fishtail			Clean
2		7.2		8.5	9.0	8.5	8.9		3		13 blows/ft Drive 1.5 sample 1.5
				9.0	9.5				4		Brown-gray mottled soft clay w/ silt stringers.
3				9.5	10.0	9.3	9.7		9		Oxidized (soil zone) CL-CH No gravel.
											Varved
				10.0	11.5			Fishtail			Clean
4				11.5	12.0	11.5	11.9		3		10 blows/ft Soft gray mottled
				12.0	12.5				4		clay CL-CH, w/silt stringers No gravel
5				12.5	13.0	12.3	12.7		6		Varved, 1/4-3/8" intervals

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
			14.0	13.0	14.5			Fishtail			Clean out
		14.0		14.6	15.0			Spoon	3	J1	8 bl/ft Drive 1.5 Sample 1.5
				15.0	15.6				3		Soft silty clay w/ 5% small gravel (CL-CH)
6				15.6	16.0	15.4	15.8		5	J6	Gray, sticky 0.1' sand layer at 14.8 to
											14.9, (SW) w/ gravel, fines.
			17.0	16.1	17.5			Fishtail			Clean out
		17.0	1'	17.5	18.0			Spoon	9		28 bl/ft Sandy gravelly clay (CL)
				18.0	18.5				14		very sandy. Good cohesion Med hd. w/
7				18.5	19.0	18.4	18.8		14		gravel to 1/2". Gray. Soft silty sand
											at 17.6-17.8.
											Lost circulation @ 14 ft. Ream hole and
											drove 6" PVC plastic casing to 17.5 ft.
											~~Casing remained in ground.~~
				19.0	20.0						Clean out

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
8				20.0	20.5	20.0	20.5	Spoon	8		36 bl/ft Same as above Sandy,
				20.5	21.0				16		gray silty clay (CL) med. hd.;
9				21.0	21.5	20.8	21.2		20		Cohesive When dry, sample parts along
											very thin silt layers at 1mm intervals (varving?)
				21.5	23.0						
10	5/8			23.0	23.5	23.2	23.6	Spoon	14		55 bl/ft Drive 1.5 sample 1.5
				23.5	24.0				22		23.0 - 24.2 - Silty firm ~~clayey~~ sandy gray
11			24.2	24.0	24.5	23.8	24.2		33		~~silt~~ clay (CL) as above. 24.2 -
		24.2									24.5. Lt gray silty clay (CL), no gravel or
				24.5	25.6			Fishtail			sand | Cleanout
12			~~25.0~~	25.6	26.1	25.7	26.0	Spoon	27	Jar	67 bl/ft - SAND - WELL SORTED, Fine to med GOOD
		~~25.0~~		26.1	26.6				35		.072 SAND, NO GRAVEL, with NUMEROUS
13				26.6	27.1	26.6	27.1		32	Jar	INTERBEDDED CLAY LAMINA (SP)
				27.1	28.0	—		Fishtail	—	—	CLEANOUT.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. 1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
14			27.5	28.0	28.5	28.0	28.4		12		Drive 1.5', sample 1.5. Firm plastic
		27.5		28.5	29.0				20		gray uniform clay (CL-CH) no sand
15				29.0	29.5	28.9	29.3		28		or gravel. Like that at 24.2-24.5
			30.0	29.5	30.4			Fishtail			
											x
16		30.0		30.4	30.9	30.4	30.8	Spoon	42		83 bl/ft Gravelly sand, graded,
				30.9	31.4				56		fine tocs to gravel (SW), little cohesion, dense.
17			31.8	31.4	31.9	31.2	31.6		27		31.8-31.9' gray firm clay (CL)
				31.9	33.2			Fishtail			Cleanout
18		31.8		33.2	33.7	33.2	33.9	Spoon	14		50 bl/ft Gray firm silty clay (CL)
				33.7	34.2				24		no sand or gravel. Drive 1.5 sample
19				34.2	34.7	33.9	34.6		26		1.5
				34.7	36.7						

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
20				36.1	36.6	36.1	36.5	Spoon	8		36 bl/ft Very silty clay (CL), gray
				36.6	37.1				18		uniform, firm ~~[illegible]~~ cohesive, some
21				37.1	37.6	36.9	37.3		18		plasticity. Drive 1.5 sample 1.5 Sample
											recov in one complete length. No gravel
			38.0	37.6	38.7			Fishtail			Clean—
22		38.0		38.7	39.2	38.8	39.4	Spoon	8		28 bl/ft Drive 1.5 recov 1.5.
				39.2	39.7				13		Silty gray ~~[illegible]~~ clay w/ trace small gravel, no
23	5/8			39.7	40.2	39.4	40.2		15		sand (CL)
				40.2	41.1			Fishtail			Cleanout
24	5/9			41.1	41.6	41.1	41.7	Spoon 2" O.D.	9	JM	41 blows/ft. Drive 1.5 sample 1.5.
25				41.6	42.1	41.7	42.3		18	JA	Recov in one length. Soft to firm silty
				42.1	42.6				23		clay (CL) w/ trace gravel as above.
				42.6	43.4	—	—	Fishtail	—	—	CLEANOUT

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO		Blows	Cont.	
26	5/9		44.2	43.4	43.9	43.9	44.1	STANDARD SPLITSPOON	12	JAR	97 blows/ft. 43.4-44.2 silty
		44.2		43.9	44.4				36		clay (CL) trace gravel 44.2 - 44.9
27				44.4	44.9	44.2	44.7		61	JAR	silt, (ML) gray #26 - CLAY
											#27 - SILT
				44.9	46.0			Cleanout			Contact is gradational
			45.5						lbs pres		
28		45.5		46.0	46.5	46.1	46.8	3" Hvorslev	420	Steel Tube	Push 3" tube ~ DRIVE 1.0' Clayey
28A				46.5	46.9	46.8	46.9		670	Jar	gravelly med-fine sand. Pebbles to 3/4", Cohesive
											Sampled 0.85', broke in tube. (SC)
				46.9	47.4						
29				47.1	47.5	47.4	47.5	3" Hvorslev	690	Jar	Would not push more than 0.1'
									(refusal)		Clayey gravelly hd sand, cohesive, Gravel
											to 1" diam.
				0.0	48.5	Reamed hole	w/	rock	bit	to	drive denison bbl.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	lbs psi		CLASSIFICATION AND REMARKS
30				48.5	49.7	48.5	49.6	Denison bbl	260	5" steel tube	Rotate long bit w/ 5" tube w/ spring steel
30A						49.6	49.7		500	Jar	catchers in end. Rotated slowly Drive 1.2
											~~Gravel knocked..~~, Clayey gravelly
											hd sand (SC). Sampled 1.2. There is
											some soft fall in debris at top of steel tube - Discard.
				49.7	50.0			Fishtail			Cleanout [grains rounded]
	gradational contact		50.5					-			Gradational change SC to CL - ... (bottom of sample)
31				50.0	51.9	50.0	51.8	Denison bbl		5" steel tube	0.2' Fallin. Rotated same scheme as above.
31A	5/9/80					51.8	51.9			Jar	Drive 1.9 Sample 1.9 V. hd gravelly
											sandy clay, compact, dense, d.gry, tight.
											Sample in good condition. Grains rounded
				51.9	53.0			Fishtail			Cleanout
32				53.0	54.8	53.0	54.7	Denison bbl			0.3 fallin Drive 1.8 Sample 1.8 Rotate
32A						54.7	54.8				as per above, Same material as above. V hd

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BØRING LØG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WCS-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				54.8	56.0			Fishtail			Cleanout
33				56.0	57.9	56.0	57.7	Denison bbl		Steel Tube	Drive 1.9 sample 1.8. Same stratum as
~~33~~A						57.7	57.8			JM	above (CL) shd. gravelly
				57.9	59.0			Fishtail			Cleanout
34	5/9		59.5	59.0	61.4	59.0-59.5	60.5-61	Denison bbl		STEEL TUBE	Rotate as above (w/catcher as above) Drive 2.4
34A		59.5				61.1	61.2			JM	Sample 1.6 Soft at 59.5-60.5.
											Soft at 61.0-61.1. Hd gray gravelly
											clay to 59.5. Sold sand 59.5 to 61.4
											Probably washed out soft sand at 59.5-60.5
											JM sample is sand (SP) uniform soft poorly
											graded qtz sand (beach sand), cohesionless.
				61.4	61.5			Fishtail			Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	lbs psi		CLASSIFICATION AND REMARKS
	5/10			61.5	61.7			3" Hvorslev	700		Pushed small tube w/ fixed piston No recovery
									drive		
				61.5	No drive			Denison bbl			5" tube w/catcher and short bit, rotated Short
											bit would not advance
35				61.5	63.9	61.5	61.9'	Denison bbl		Jar	5" tube w/catcher w/ long bit. 3 Jar samples
36						61.9	62.3			Jar	of very wet loose cohesionless well sorted
37						62.3	62.5			Jar	medium grained sand (SP) [beach sand]
											Drive 2.4 recov 1.0.
38				63.9	64.4	63.9	65.9	3" Hvorslev	340	STEEL TUBE	Push: 2.1 Recov. 2.4 (some fall in)
			2.1	64.4	64.9				460		Disregard top 1.5 feet, prob. disturbed.
				64.9	65.4				540		(SP) cohesionless gray uniform ~~fine~~ fine grained sand
				65.4	65.9				540		
38A				65.9	66.0	65.9	66.0		720	Jar	
				66.0	67.9						

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BORING LOG
FIELD DATA

Project ____________ Site ____________ Date ____________
Location ____________ Job No. ____________
Drill Rig ____________ Inspector ____________ Operator ____________ Surface El ____________ Boring No. ____________

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	blows / 0.5'	Container	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
~~39~~				67.7	68.2			2" O.D Split Spoon	78		145/0.7'. Drive 0.7 sample 0.4
39				68.2	68.4	68.0	68.4		67	Jar	Med. sand (SP) cohesionless as above
				68.4	70.5'			Fishtail			Cleanout
				70.5	71.0				74		100/0.5' Med. grained SP sand as
40				71.0	71.1	70.7	71.1		26		above
				71.1	73.5			Fishtail			
41				73.5	74.0				47		112/0.75' Med to fine (SP) sand as above.
				74.0	74.25	73.8	74.2		65		
				74.3	76.5			Fishtail			
42				76.5	77.0				42		92/0.70 (SP) fine sorted sand as above.
				77.0	77.2	76.9	77.1		50		

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-1

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	blows		CLASSIFICATION AND REMARKS
				77.2	79.5						
43				79.5	80.0	79.7	80.0	2" O.D. SS	47	Jae	97/[illegible] Sand (SP) med from 79.5 to 80.0
44				80.0	80.2	80.0	80.2		50	Jae	Fine to v. fine 80.0 - 80.2
				80.2	83.5			Fishtail			
45				83.5	84.0	83.6	84.1	2" Splitspoon	70	Jae	100/0.6' Sand (SP) fine grained cohesionless
				84.0	84.1				25		[beach sand] as above.
			85.0	84.1	86.0			Fishtail			~~Cleanout~~ Hard at 85' ~~v. hd~~
											~~gravelly silty clayey sand.~~
46		85.0		86.0	86.1	86.0	86.1		50		v. hd gravelly clayey sand, cohesive.
											(SC)
47	5/10			86.0	88.4	86.0	87.6	Denison bbl		Cardon tube	5' tube w/catcher, lmabit V. hd sandy grav-
47A						88.3	88.4			JAE	elly clay Angular and rounded gravels (glacial till) (CL)

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

hole grouted
88.4 - 0.0

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BORING LOG

FIELD DATA

Moved location from original site because of low hanging high voltage power lines.

Project DETROIT SEWER Site ____ Date MAY 5-, 80

Location STA 125+ ' , ' E of ℄ Job No. ____

Drill Rig ____ Inspector MURPHY Operator HARRIED Surface El 608.8 Boring No. WES-2

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	# blows	CONTAINER	CLASSIFICATION AND REMARKS
		0.0									
	5-5			4.0	4.5	—	—	2" O.D. Split spoon	2		140 lb hammer — No sample taken. Fill
				4.5	5.0				3		material. Land fill (dump) w/ assorted
				5.0	5.5				2		waste, little or no soil.
			36.7	5.5	36.7			4 3/4" Fishtail			Cleanout
1	5-6-80	36.7		36.7	37.2	36.7	37.4	2" S.S.	16	Jar	75 blows/ft. Gray silty clay (CL),
				37.2	37.7				30		firm, somewhat plastic. Drive 1.5, recov.
				37.7	38.2				45		1.5 No sand
				38.2	39.2			Fishtail			Cleanout
				38.7	39.2			2" S.S	5		24 blows/ft. Gray soft silty clay
2	5/6			39.2	39.7	39.2	40.0		11	Jar	CL-CH, plastic. Drive 1.5 sample 1.0
				39.7	40.2				13		(0.5 fall-in). No sand

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows		CLASSIFICATION AND REMARKS
				40.2	41.9						
				41.9	42.4			2" S.S.	5		25 blows/ft. Gray soft plastic clay
3	5/6			42.4	42.9	42.4	43.0		11	Jm	(CL-CH) as above. No sand.
				42.9	43.4				14		Drive 1.5 rec. 1.5' (compressed in sampler
				43.4	44.5			Fishtail	?		Cleanout.
											Stratum grades more plastic w/ depth
				44.5	45.0			2" O.D. SS.	31		17 blows/ft Soft gray plastic clay
4				45.0	45.5'	45.0	46.0		7	Jm	as above (CH), Compressed in sampler
				45.5	46.0				10		No sand.
				46.0	47.0						
			47.2	47.0	47.5'	1		2" S.S.	7		31 b/ft. Firm silty gravelly clay (CL).
5		47.2		47.5	48.0	47.5	48.5'		14		cs. sand to pea gravel. Compressed in sampler.
				48.0	48.5				17		

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				48.5	49.3			Fishtail			Cleanout
				49.3	49.8			2" O.D. S.S	8		39 b/ft Drive 1.5' recov. Soft to
6	5/6			49.8	50.3	49.8	50.7		14	Jar	firm gravelly sandy silty clay (CL)
				50.3	50.8				25		cs sand and fine gravel.
				50.8	51.5			Fishtail			Cleanout
		51.3									Harder at 51.3 - (Uniform gray silt).
			51.7	51.5	52.0			2" S.S.	35		102 b/ft Gravelly sand, gray
		51.7		52.0	52.5				51		(SW) med. grained, hd but friable (some
7	5/6			52.5	53.0	52.5	53.0		51	Jar	cohesion) (Uniform gray silt 51.5 to 51.7)
											Sample is moist, not wet
			53.5	53.0	54.0			Fishtail			Cleanout
		53.5		54.0	54.5			2" S.S.	17		200 blows/ft V. hd silty gravelly
8				54.5	55.0	54.8	55.3		75		clay (CL) 5-10% gravel (small)
				55.0	55.5				125		

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				55.5	56.7			Fishtail			Cleanout
				55.7	56.2			2" O.D. S.S.	33		208 blows/ft V. hd gravelly silty
9				56.2	56.7	55.7	56.2		100	Jar	clay as above (CL), dk gry.
				56.7	57.0				54 lb 2.25-refusal		
				57.0	58.0			Fishtail			Cleanout
10			58.6	58.0	58.5	58.2	58.6	2" O.D. S.S.	27	Jar	Drive 1.0 270 blows/ft
11		58.6		58.5	59.0	58.6	58.8		135 refusal	Jar	58.0 - 58.6 - hd clay, silty (CL) as above
											58.6 - 59.0 Fine to med. sorted qtz sand, friable,
											dense (SP) dk gray, some cohesion - One large
				59.0	59.5			Fishtail			cobble separates the two strata at 58.6'
.											
12				59.5	60.0	59.5	60.1	2" O.D. S.S.	77	Jar	Drive 0.9 (100 blows/0.4' = 250 blows/ft)
				60.0	60.4				100		Dense gray fine grained loose (cohesionless) sand (SP) as
											above [beach sand]. Well sorted, qtz sand.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				60.9	61.5			Fishtail			
13	5/6			61.5	62.0	61.5	61.9	2" S.S.	49	Jar	75/0.3' : 250 blows/ft. Drive 0.8.
				62.0	62.3				75		Uniform gray silt (ML) [fine laminae in bench
											sand stratum], silt from 61.5 to 62.0
		Stratum has v. fine to silt layers within it.									Sand, fine (SP) 62.0 to 62.3. Sample 13
											is silt (ML). Gradation is fining.
				62.3	63.5						Cleanout
14				63.5	63.9	63.5	63.6	2" S.S.	100		Drive 0.4' (100 blows/0.4) = 250 blows/ft
											Uniform gray well sorted fine qtz sand (SP)
											as above. Dense but cohesionless
				63.9	64.5			Fishtail			Cleanout
				64.5	64.9			2" O.D. SS	100 refusal		Drive 0.4 No sample

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ELEVATION [illegible] 08.0 [illegible]

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	1980 DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows.	CONT	CLASSIFICATION AND REMARKS
				64.9	65.5	—		FISHTAIL	—		CLEANOUT
15	5/7			65.5	65.9	65.5	65.9	STANDARD SPLITSPOON	10/	JAR	SAND – Uniform, grey, well sorted, dense fine gr. SAND, cohesionless. DRIVE 0.4'
				65.9	66.5	—		FISHTAIL	—		CLEANOUT
16.	5/7			66.5	66.95	66.5	66.95	STANDARD SPLITSPOON	100	JAR	SAND – SEE ABOVE – DRIVE 0.45'
				66.95	68.0	—		FISHTAIL	—		CLEANOUT.
17	5/7			68.0	68.5	68.0	68.75	STANDARD SPLITSPOON	47	JAR	100/0.25' - 400 blows/ft Sand as
				68.5	68.75				100		above.
				68.75	68.8	—		FISHTAIL	—		CLEANOUT
				68.8							

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows		CLASSIFICATION AND REMARKS
18	5/7/80			68.8	69.3'				53	Jan	100/0.3' V. fine sorted qtz sand (SP)
				69.3	69.6	69.2	69.5		100		
				69.6	71.0						
19				71.0	71.5	71.2	71.5	2" O.D. SS	100	Jan	100/0.5 = 200 blows/ft Fine well
											sorted qtz sand (SP) as above
				71.5	72.5						Cleanout
20				72.5	73.0	72.8	72.9	2" O.D. SS	102	Jan	102/0.5 = 204 blows/ft Fine to v. fine
											sand SP as above
21				73.0	74.5						Cleanout
21				74.5	74.9	74.7	74.8	2" O.D. SS	110	Jan	110/0.4 0.1' recov. (SP) Fine sand as above
				74.9	76.0						Cleanout

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	Blows		CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
22				76.0	76.5	76.3	76.5	2" O.D. Split Sp.	90		115 blows/0.6' (SP) fine sand well
				76.5	76.6				25		sorted as above
				76.6	78.2						Cleanout
				78.2	78.7			S.S.	90		140/1.1' No sample
				78.7	78.8				50		
				78.8	79.5			Fishtail			Cleanout
23				79.5	80.0	79.6	79.9	2" OD S.S.	105		105/0.5' · 210 blows/ft. Fine
											sand (SP) as above
				80.0	81.5				~~[illegible]~~		~~[illegible]~~
				81.5	84.85				110		110/0.35 (refusal) No sample
				/							
				84.85	85.0						Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				85.0	85.6			2" O.D. Split Spoon	44		160 bl/ft v. fine sand (SP) or above
24	5/7/80			85.6	86.1	85.5	86.1		80		and silt (ML), uniform. Grades finer
											and coarser gradually, all part of same
											stratum. Generally borderline between
											silt and sand size.
			86.5	86.1	87.0			Fishtail			Cleanout
~~25~~		86.5		87.0	87.5			2" O.D. S.S.	75		60/0.25' drive ~~125~~ 0.75' Graded qtz sand w/ gravel
25				87.5	87.75	87.2	87.7		60		to 1/4" (SW) gray, dense. Med to coarse
											Outwash sands and gravels?
			88.4	87.8	88.5						Cleanout
		88.4		88.5	89.0				74		126/0.6 Silty gravelly sand, compact,
26				89.0	89.1	88.6	89.0		50		good cohesion. (SC-SM). Probably associated
											w/ till environment below
				89.1	90.0						

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows		CLASSIFICATION AND REMARKS
27				90.0	90.5	90.2	90.5	2" O.D. S.S	137	Jar	137/0.5' Clayey silty gravelly sand, hd,
											dense. (SC-SM). Subrounded pebble and cs
				90.5	91.0						sand.
											Cleanout
~~28~~				91.0	91.5				83		128/0.7 Same stratum (probably
28				91.5	91.7	91.2	91.7		45	Jar	outwash channel cut into till plain)
											SC-SM hd.
					Bottom of hole 91.7'.						
					Grouted w/ __ bags calcium aluminate cement						

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El 609.8 Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		0.0		0.0	10.1	-	-	Fishtail			Fishtailed, cleaned to 10.1' through surface
											debris and recent fill.
				10.1	12.5			5" Hvorslev	320		Drive 2.4 No sample [illegible] in old
									430		landfill (garbage) [illegible]. Drive 8' steel
									470		casing to 8.4'
			28.7	12.5	29.0				440		Land fill, loose assorted refuse.
1	4/29	28.7		29.0	29.2	29.0	29.2	5" Hvorslev		[illegible]	Drive 0.2 to refusal. Clayey gravelly sand
											(SC), dk gray, soft, sticky [illegible]
											pebbles, cobbles to 2" diam.
				29.2	30.0			Fishtail			Cleanout. Pulled casing; rock bit to
											30.0 ft (8'). Drive 30' steel 8" casing.
2	4/30/80			29.6	30.1	29.6	30.0	5" Hvorslev			Drive 1.4, no recov. [illegible].
				30.1	34.6						[illegible] classification
				30.6	31.?						Sandy gravelly clay, med. stiff. Gravel 5%,
											1/16" - 1/4" diam rounded pebbles. Dk gray (CL)

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				31.0	31.5		-	Fishtail	380		Cleanout
									560		
3				31.5	32.0	31.7	31.9	5" Hvorslev	380	JAR	Used smooth tube (sawedge removed). Drive 0.9
				32.0	32.4				[illegible]		Recov. 0.5 good sample. Gravelly clay (CL),
											med. stiff. More plastic toward bottom
				32.4	33.0			Fishtail			Cleanout / Gradually less sandy (gritty) from 30.0 to 31.5
				33.0	35.4			Denison bbl, 5"		NONE	0.1' stickup. Lost sample. Drove with 6" carbaloy
											bit below tube. Will redrive w/ short bit.
4				33.0	35.8	33.0	33.9	Denison bbl, 5" Tube		Tube	Rotab. Recov. 2.1, Drove 2.8. Gravelly, gritty clay
4A						33.9	34.0			JAR	dk gray med stiff (CL) to 34.6, then plastic
5			34.6			34.0	35.0	"		Tube	non gritty, fat clay (CH) below 34.6, dk gray, med hd,
5A		34.6				35.0	35.1			JAR	very little gravel (<5%) [Note: Samples 4 and
											5 were overcored and are ~~probably~~ disturbed!]
				35.8	36.5	-		Fishtail	-	-	Cleanout

34.6

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______

Location ______ Job No. ______

Drill Rig ______ Inspector ______ Operator ______ Surface El 509.6 Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
6				36.5	38.7	36.6	37.5	Denison bbl 5"		Tube	Rotate carbaloy bit above tube Drive 2.2
6A						37.5	37.6			Jar	Recov 2.1. dk gry fat clay (CH) med.
7						37.6	38.6			Tube	hd, very little gravel (<1%). Uniform color
7A						38.6	38.7			Jar	and texture thruout. Samples in very good
											condition.
				38.7	39.9	-	-	Fishtail	-	-	Cleanout.
8			40.3	39.9	42.2	39.9	40.0	Denison bbl		Tube	Rotated same as above Drive 2.3 sample 2.3
8A		40.3				40.0	41.1			JAR	39.9 - 40.3 CH as above. 40.3 - 42.2
9						41.1	42.1			TUBE	(CL) silty clay, dk gry, med hd. Samples in good condition
9A						42.1	42.2			JAR	except 39.9 - 40.3, which is somewhat disturbed,
											soft. Little or no gravel Laminae are
				42.2	43.0	-	-	Fishtail	-	-	Cleanout | distorted, as in turbulent sediment conditions (wave action?)
10				43.0	45.4	43.0	44.1	Denison bbl			Rotate as above. Drive 2.4, sample 2.4
10A						44.1	44.2				Sticky clay CL-CH 5% small gravels Grav.
11						44.2	45.3				to ½" diam
						45.3	45.4				

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				45.4	46.0						Cleanout
12				46.0	48.4	46.0	47.1	Denison bbl		Tube	Drive 2.4 Recd 1.5 Rotated as above
12A						47.1	47.2			Jar	46.8-47.0 Soft fine, sand SP. Rest of
13						47.2	47.7			Tube	sample is med. hd silty dk gray clay w/
13A						47.7	47.8			Jar	5% gravel to ½" diam. (CL-CH)
				48.4	49.4			Fishtail			Cleanout
14			50.4	49.4	51.7	49.4	50.7	Denison bbl			Drive 2.3 Recd 1.9 Rotated as above
14A						50.7	50.8				49.4-50.4 CL-CH med hd to soft clay.
15						50.8	51.3				50.4-51.4 Silty-clayey gravelly
15A						51.3	51.4				sand (SM-SC) w/ 15-20% gravel to 1",
											rounded pebbles of igneous rock. Poorly sorted - angular to rounded grains (outwash?)
											Top 0.4' of # 14 disturbed - #15 broke
											short to 0.5' length. CLAY-SAND
											contact sharp, horizontal.

CLAY


WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
16				51.7	52.0						Cleanout
16A											
17	16			52.0	54.1	52.0	52.9	Denison bbl		Tube	Drive 2.1 Recov. 1.3 [illegible]
17A	16A					52.9	53.0			JAR	gravelly clayey firm sand (SC-SM) 15-
[illegible]	17					53.2	53.3			JAR	20% gravel to 1½" diam. Uniform silt (ML)
	~~17A~~										light gray, firm 52.9-53.2, sharp
											contact w/ sand above & below, stratified,
											no gravel, no sand w/some clay laminae #17 is sand, #16A
											is silt, #16A sand w/silt on bottom.
			54.5	54.1	55.0						Cleanout
5/1/80	18	54.5		55.0	57.4	55.0	55.5	Denison bbl		Tube	Rotate as above Drive 2.4 Sample 1.8
	18A					55.5	55.6			Jar	56.8-56.7 discarded. Med. hd silty
	19					55.6	56.7			Tube	clay (CL) dk gray w/occasional thin laminae
	19A					56.7	56.8			Jar	of gray silt (ML). 5-10% pebble gravel, rounded.
											Sample break w/effort along silt laminae.
				57.4	58.2						Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES 3

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
20				58.2	59.8	58.2	59.0	Denison bbl		Jar	Drive 1.6 sample 1.4
21		59.0				59.0	59.6			Jar	Fine silty dk gray dense sand w/trace
											gravel. (SM) from 59.0-59.6. Soft
											clayey sand (SC) from 58.2-59.0, compressed
											in tube. Sample [illegible]
				59.8	60.0			Fishtail			Cleanout
22				60.0	61.0	60.2	60.3	Denison bbl		Jar	Short bit rotated as above. Drive 1.0, sample 1.0
22A						60.9	61.0			Jar	Gray hd dense silt (ML) w/occasional
											gravel (<5%) and fine sand laminae, distorted
											60.9 to 61.0. From 60.0-60.9 is fine
											silty sand SM-SC. Contact is sharp.
											Sample eroded badly, could not wax too
											disturbed.
				61.0	61.5			5½" Rockbit			Cleanout Much gravel

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Sheet 6 of ______ Sheets

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El 609.8 Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
23				61.5	63.9	61.5	62.5	Denison bbl		Tube	Long bit, rotated as above Soft at 62.6
23A						62.5	62.6			Jar	Recov. 2.0 Drive 2.4 Interbedded ~~fine~~
24						62.6	63.4			Tube	~~to~~ very fine sand, well sorted (SP) at 2 sand
24A						63.4	63.5			Jar	~~dk~~ silt, firm to soft, dense (ML), all
											dk to lt. gray. Two tube samples were in
											in very good condition. No gravel.
				63.9	64.4						
											Loose sand at 64.4'
				64.4	66.8				200 lbs		Rotate long bit as above. Drive 2.4 sample
											Drove fast (soft soil). Lost sample in hole. Recovered
											w/ split spoon to sample. Soil so loose it would
											not stay in split spoon. Probably v. loose sand
											(SP) as noted in 7B boring 4 and 5'. Probably
											should assign minimum strength.
				66.8	67.0			Tech bit			Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	lbs pres.		CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				67.0	67.5			3" Hvorslev	500		Pushed 3" tube in loose sand
				67.5	68.0				600		chuck slipped at 68.4, at 68.7. Recov. only
25				68.0	68.5				660	Jar	1" of sample. v. loose noncohesive fine gr.,
				68.5	69.0				670		uniform sorted qtz sand (SP). Very wet.
26				69.0	69.1				740	Jar	Went in w/ 2" split spoon to retrieve sample
				69.1	69.5			Fish			
27				69.5	71.9	69.5	69.9	Denison bbl.	200lb	Jar	Installed 5" tube w/ spring steel catcher.
28						70.2	70.6			Jar	Rotated w/ long bit, bypassing water. Still
29						70.7	70.9			Jar	soft or loose. Drv. 2.4 recov. 1.4. Sample
											recovered but in poor condition. Took 3 jar
											samples, no waxed tube. V. wet, cohesionless
											well sorted qtz. sand (beach sand) (SP), fine
											grained. Very uniform throughout. Positive reaction
											to dilatancy test.
				71.9	72.0			Fish			Cleanout

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Sheet 8 of ______ Sheets

BORING LOG

FIELD DATA

Project ______ Site ______ Date ______

Location ______ Job No. ______

Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
30				72.0	72.5	72.0	73.0	3" Hvorslev	510	Steel Tube	5" tubes will not hold material well enough.
				72.5	73.0	-			690		Running 3". Drive 1.9 sample 1.1.
30A				73.0	73.5	73.0	73.1		690	Jar	Sample left in steel tube, disturbed when breaking
				73.5	73.9				794 [illegible]		down sampler. v. loose wet sand SP as
											above, v. fine grained. This stratum is
											getting finer w/ depth. No gravel.
				73.9	74.0						
31				74.0	76.1	74.4	76.0	Denison bbl		Steel Tube	Rotating w/catcher. Kept sampler vertical
31A						76.0	76.1			Jar	when breaking down, capped both ends before
											laying down. ~~[illegible] harder at 76 ft.~~ Drive
											2.1. Recov. 1.7. Left in 5" steel tube. Waxed
											top, plugged bottom. Sample appears in good
											condition. 0.4' unaccounted for. Possibly washed
											away from top
				76.1	77.0			Fishtail			Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	lbs psi		CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
32	5/2/80			77.0	79.4	79.0	79.4	Denison bbl	240	Jar	Rotating 5" tube w/long cutting bit and catcher
											Drive 2.4, recov 0.4'. Probably washed out.
											Replacing long bit w/ short bit on next drive
				79.4	80.0			Cleanout			SP same as above.
				80.0	81.4			Denison bbl			Drive 1.4. Rotated (tube w/catcher) as above.
33										JAR	Recov. only 0.3'. Probably washed out
				81.4	82.0			Fishtail			Cleanout
34	5/2/80			82.0	82.5	82.0	84.2	3" Hvorslev	400	STEEL TUBE	Pushed 3" tube. Drove 2.3, sample 2.3
				82.5	83.0				510		Fine qtz sand, sorted, (SP) [beach sand]
				83.0	83.5				580		As above. Not quite as fine grained as above.
				83.5	84.0				650		Loose, wet. Subangular grains
34A				84.0	84.3	84.2	84.3		700	JAR	
				84.3	85.0			Fishtail			Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
35				85.0	86.5	85.0	86.9	3" HVORSLEV	350		Drove 2.0. Sample 2.0
				85.5	86.0				500		V. fine sorted quartz sand (SP) as above.
				86.0	86.5				600		Loose, v. wet, cohesionless.
35A				86.5	87.0	86.9	87.0		710		
											Using small tubes because they hold sample well.
			87.2	87.0	87.5						Lost water temporarily at 80.0 ft while
											cleaning out.
36	5/3/80	87.2		87.5	88.0	87.5	88.7	3" Hvorslev	360	Steel Tube	Pushed 3" tube. Drove 1.5, recov 1.4
				88.0	88.5				440		Uniform gray qtz silt (ML), dense, compact
36A				88.6	89.0	88.7	88.9		refusal 600	Jay	but easily friable — (low cohesion). Same
											stratum as above, fining w/depth
			89.3	89.0	89.5			Fishtail			Cleanout [STRATUM CHANGE]
37		89.3		89.5	90.0	89.5	89.7	3" Hvorslev	580	Jay	Drove 0.7 recov. 0.5. Clayey gravelly
38				90.0	90.2	90.1	90.2		500 refusal	Jay	fine to ~~med~~ coarse sand, well graded. Gravel 20%
											up to 1/2" diam., rounded to subangular pebbles. Firm to hard,
											w/ good cohesion. (SC)

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-3

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
39	5/3/80			90.0	92.4	90.0	92.0	Denison bbl (5")		5" STEEL TUBE	Long bit, tube w/ catcher. Drove 2.4', sample
39A						92.1	92.2			Jar	2.2'. Hd dense gravelly clayey
											sand as above. Subangular to subrounded gravel and
											coarse sand in clayey sand matrix.
											(SC). [GLACIAL TILL]. Seems more sandy,
											a little less hard than till in WES-4
40				92.4	94.8						Drove 2.4, no recovery. Drilled hd,
40A											but must have slipped out of tube
				Bottom of hole @ 94.8							
				HOLE COMPLETED				MAY 3 / 80.			Grouted w/ bags of Calcium
					aluminate	cement		grout	to	30.0	ft depth (elev 580)

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LID OF TUBE IS BOTTOM

Kelly push 2.35

BORING LOG
FIELD DATA

Project DETROIT SEWER Site ______ Date 1/22/80
Location STA 112+81, 18 FT E of ℄ ELEV 606.3 Job No. ______
Drill Rig Failing 1500 Inspector MURPHY Operator B. HARRIED Surface El ______ Boring No. WES 4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
1	4/22	0.0	0.2	0.0	0.5	0.0	0.9	5" I.D. Shelby	140	S.T	Silty sand, organic material, roots
1A		0.2	1.65	0.5	1.0	0.9	1.0	JAR	170		Fine sand (SP), w/ limonite stain
2		1.65	~~3.0~~	1.0	1.5	1.0	1.9	5" I.D. Shelby	170	S.T	Silt, tan (ML)
2A				1.5	2.0	1.9	2.0	JAR	245		"
				2.0	2.35	-			520		
				2.35	3.0			6" Fishtail			
3		2.5	6.0	3.0	3.5	3.0	4.2	Hvorslev	225	Tube	Clayey silt (ML) w/ gravel.
3A				3.5	4.0	4.2	4.3	JAR	250		No mud, 0.0 to 6.0
4				4.0	4.5	4.3	5.3	Hvorslev	350	Tube	Used Mud, 6.0 to bottom.
"				4.5	5.0				360		
4A				5.0	5.35	5.3	5.35	JAR	425		
				5.35	6.0			6" Fishtail			
		6.0	6.9	6.0	6.5	~~7~~			75		Wet compressible gravelly clay (CL) (compressed in
5A				6.5	7.0	6.6	6.7	JAR	140		Recov. 1.5 (8.4-6.9) tube, no recov.)
5		6.9		7.0	7.5	6.9	8.3	Hvorslev	180	Tube	Dk gray clay (CL-CH) w/ thin stringers of

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AD-A096 440 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY, DETROI--ETC(U)
JAN 81 D ALBERT, G C HOFF, B LORENCE NCE-IA-80-055
UNCLASSIFIED WES/TR/GL-81-2 NL

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AD
A096440

BORING LOG
FIELD DATA

Project ______ Site ______ Date 4/23/80
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				7.5	8.0				220		med grained, hd, limonite stained sand (tube sample broke along
6A				8.0	8.4	8.3	8.4	JAR	230		two of these sand stringers.
											Note 6.0 to 6.9 is wk. gravelly clay that
				8.4	9.0	—	—	Fishtail	—		caved in hole when we came out
											No SAMPLE #6
	4/23/80						Clean out to 9.0. Quick gel mud.				
7				9.0	9.5	9.0	10.1		100	TUBE	Dk gray clay w/gravel<5%. (CL-CH)
7A				9.5	10.0	10.1	10.2		120	JAR	Soft, 9.0-10.2, disturbed when extruded, v/wet.
8		SAME		10.0	10.5	10.2	11.3		115	TUBE	Stiff, 10.2-11.4, in good condition.
				10.5	11.0				190		
8A				11.0	11.4	11.3	11.4		220	JAR	
				11.4	12.0			Fishtail —	—	—	— CLEAN OUT
											Push 2.4, 1' sticking, sample only 1.4
9				12.0	12.5	12.0	13.3	Hvorslev	170	TUBE	Dk gray stiff CL CH clay, 5% gravel (small
				12.5	13.0				210		Sample broke in two at 12.6.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				13.0	13.4	13.3	13.4	HVORSLEV	240	JAR	Clay breaks along horizontal, rough planes
											1/4" thick when broken by hand (bedding). Dry sample
9A				13.4	14.0			FISHTAIL			very well stratified (varved), w/ much v. fine gravel (see jar)
											Cleanout to 14.0, taper hole
10		14.0	16.0	14.0	14.5	14.1	15.2	HVORSLEV		TUBE	0.1 STICKUP CL-CH Gray sticky clay
				14.5	15.0						w 10% gravel < 1/8" diam.
10A				15.0	15.5	15.2	15.3			JAR	
11				15.5	16.0	15.3	16.2			TUBE	
11A		16.0	20.9	16.0	16.3	16.2	16.3			JAR	Gritty sandy clay (CL-CH) w/ pebble-
											sized gravel 10%, or clayey sand (SC)
											Very sandy
				16.3	18.0			FISHTAIL			CLEAN OUT
				18.0	18.5			Lost sample from tube.			Removed Hvorslev sampler, used 2" split
				18.5	19.0			spoon to recover sample.			Pushed spoon.
11B				19.0	19.5					JAR	Gritty sandy clay w/gravel as above, very
				19.5	20.0	18.5	18.8	SPLITSPOON PUSHED			sandy.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
	4/23			20.0	20.4						
				20.4	20.8			FISHTAIL			CLEANOUT
12		20.9		20.9	21.5	21.0	21.9	Hvorslev 5"		TUBE	Material hard to push. Refused at 22.3
12A				21.5	22.0	21.9	22.0	Left 0.3 in		JAR	V. stiff dk gry clay (CL/CH) w/ occasional small gravel,
				22.0	22.3			hole			5% gravel. Breaks along hor. planes w/ difficulty
				22.3	23.2			Fishtail			Cleanout
13				23.2	26.3	25.3	26.3	Denison barrel		5" TUBE	Rotating to drive sampler (5" tube) 6" O.D. Carboloy bit
13B						25.1	25.3	"		JAR	Hd stiff clay CL - v. little small gravel
13A						26.2	26.3	"		JAR	13B has larger gravel up to 1¼" various pebbles.
											Ran 600 lbs pressure while rotating
				26.3	27.0			Fishtail			Clean out

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
14				27.0	29.4	27.0	28.0	Denison barrel		TUBE	Rotated Denison barrel. Sample stayed in
				Left 1.4' in hole, broke along horizontal							hole. Went in again w/ denison barrel, overcored,
				bedding plane.							came out again.
14A						28.0	28.1	Denison		JAR	27.0-28.0 v. hd dk gray silty clay (CL) w
											<5% gravel (1/8")
				29.4	30.0			FISHTAIL			Clean out to 30.0
				30.2	32.5	31.8	32.4	Denison bbl	80 lb	TUBE	Rotated Denison bbl.
				Left 1.6 in hole, broke along sandy horiz							
				plane		32.4	32.5			JAR	

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BORING LOG
FIELD DATA

Project ___ Site ___ Date ___
Location ___ Job No. ___
Drill Rig ___ Inspector ___ Operator ___ Surface El ___ Boring No. WES 4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				32.5	33.0						
16	4/24			33.0	35.4	33.0	33.9	Denison bbl		Tube	Rotate w/ Carbaloy bit Hd dk gry clay
						33.9	34.0			JAR	stiff (CL) w/ <5% gravel to 1"
											Lost water in this drive (33-35')
											Modified bit to place mouth of tube below teeth
											of bit
				35.4	36.0			Fishtail			Cleanout
17				36.0	38.4	36.0	37.2	Denison bbl, 6" bit	700lb		Hd stiff dk grn-gry clay (CL) w/ 5% gravel
17A						37.2	37.3				to 3/4" dia.
				38.4	39.0	-	-	Fishtail	-	-	Cleanout. Bbl returned to 38.4 (0.5
											stickup. So drive 1.8)
18				39.0	40.8	39.0	39.6	Denison bbl		Tube	Drive 1.8, too hd. Fishtail went back in to 40.8
18A						39.6	39.7			Jar	Sample 1.3. Hd gray clay CL w/some gravel to
19						39.7	40.2			Tube	2" diam. Sand stringers to 1/2" thick occasionally

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
19A	4/24					40.2	40.3	Denison bbl		JAR	Sample broke at 39.7, so tubed two samples.
				40.8	41.5			Fishtail			
			40.2								
		40.2		41.5	43.9			5" Hvorslev			Drove 2.4 Softer, lost sample. In sand.
				41.3	43.7			" "			No resistance - No sample (Sand)
				43.7	44.7			Fishtail			Cleaned to 44.5'
20				44.7	45.0	44.7	45.8	5" Hvorslev	520		Drive 1.3 lost 0.1' in hole
20A				45.0	45.8	45.8	45.9		720		Dk gry clayey fine sand (SC), soft, uncemented,
				45.5	46.0				Refused		w/ 20% gravel 1/8-3/4" diam.
				46.0	46.5						Large gravel refused pushed tube

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BORING LOG
FIELD DATA

Project ________ Site ________ Date ________
Location ________ Job No. ________
Drill Rig ________ Inspector ________ Operator ________ Surface El ________ Boring No. ________

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
21				46.5	48.6	46.5	47.4	Denison bbl		Tube	6" Cookatong bit Drove 2.1 Recov. 1.8
21A						47.4	47.5			Jar	Dk gray firm gravelly sand 20% gravel
22						47.5	48.2			Tube	up to 2½" (rounded cobble), some clay in
22A						48.2	48.3			Jar	matrix (SP or SC)
											Drive 2.1 sample 1.8, left .3 in hole
											Sample broke into short lengths because of large gravel
				48.6	49.0			Fishtail			Cleanout
23			49.5	49.0	51.4	49.0	~~49.~~49.9	Denison bbl		Tube	Drove slowly "Sand as above to 49.5, then soft clay
23A (*)		~~49.5~~				49.4	50.0			Jar	23A not representative of 23. (*)
24		49.5				50.0	51.2			Tube	Dk gray hd clay w 5% gravel (CL)
24A						51.2	51.8			Jar	(Contact clay & sand in #23 @ 49.5')
				51.4	52.0	-	-	Fishtail	-	-	Cleanout
									!		
25				52.0	54.4	52.0	53.0	Denison bbl	200 lb	2	Drove 2.4 recov 1.4 Lost 1.0' @ 53.1-54.1
25A						53.0	53.1				Hd dk gray silty clay CL w/ 5-10% pebble gravel

26 54.1 54.3
26A 54.3 54.4

Sheet 5 of ________ Sheets

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Lbs pres.		CLASSIFICATION AND REMARKS
				60.5	62.9	Lost	sample	Denison bbl	40		Rotated slowly w/ carboloy bit. Lost sample
											when pulled out. Will try pushing w/ 3" tube
29				62.8	63.6	62.8	63.3	3" Hvorslev		3" steel tube	Pushed to refusal. Hd gry gritty clay CL,
29A						63.3	63.35			Jar	63.0-63.3?, probably sand 62.8-63.0
				63.6	64.0			Fishtail			Cleanout
30				64.0	65.9	64.0	65.0	Denison bbl	780	Tube	Rotated w/ carboloy bit. Let tube stick out 1/2"
30A						65.0	65.1			Jar	past teeth. Fine silty light-med gray sand,
											occasional gravel <5% (SM) Sample in
											good condition. Drove 1.9 sampled 1.1, lost 0.8'
				65.9	66.5	-	-	Cleanout	-	-	Fishtail
31				66.5	68.2	66.5	66.8			Tube	Drove 1.7 Uniform fine gray sand (SP), soft
31A						66.8	66.9			Jar	Sample 0.4. Sample soft.

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES 4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	lbs pres.		CLASSIFICATION AND REMARKS
32				69.0	69.5	69.0	70.0	3" Hvorslev	480	3" steel tube	Pushed 3" tube in sand, Drove 1.3, samp. 1.1
32A				69.5	70.0	70.0	70.1		620	Jar	Fine uniform gray qtz. sand (SP) no gravel
				70.0	70.3				REFA		Probably beach sand (well sorted, uniform)
					7.						
				70.3	70.5			Fishtail			Cleanout.
34				70.5	70.9	70.5	70.9	3" Hvorslev		Jar	Drove 0.4 to refusal , Same as above
											(SP)
				70.9	71.8			Fishtail			Cleanout
35	4/28			71.8	73.1	71.8	72.0	Denison bbl	780	Jar	6" Carboloy bit. Drove 1.3 sample 0.4
											V. soft uniform fine qtz sand (SP) sample outer barrel
											too disturbed to wax in tube. Jar only.
				73.1	73.5	-	-	Fishtail			Cleanout
36				74.5	76.4	74.5	76.3	3" Hvorslev	440 640	3" steel tube	Pushed 3" tube Same as above SP soft
36A						76.3	76.4		700 740	Jar	Drove 1.9 recov 1.9

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
				76.4	77.0	-	-				Cleanout
37	4/26			77.0	78.9	77.0	78.7	3" Hvorslev	400	3" Steel Tube	Drive 1.9 sample 1.8. Same as above
				77.5	78.0				670		fine uniform gray qtz sand (SP)
37A				78.0	78.5	78.7	78.8		750	Jar	
				78.5	78.9	"			800		
				78.9	79.7	-	-	Fishtail			Cleanout
38				79.7	80.2	79.7	80.7	3" Hvorslev	500	Steel Tube	Drive 1.2, chuck slipped, could not redrive
				80.2	80.7				690	"	Soft sand, fine (SP) changing gradually to
38A				80.7	80.9	80.7	80.8		800 (refusal)	Jar	very fine sand at bottom. Gradational fining of
											sand toward bottom. Recov 1.1, Drive 1.2
				80.9	81.5			Fishtail			Cleanout

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES 4

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
39				81.5	82.0	81.5	82.7	3" Hvorslev	470	steel tube	Drive 1.4 Rec. 1.3. Very fine gray
				82.0	82.5				730		soft uniform qtz sand (SP) [beach sand]
39A				82.5	82.9	82.7	82.8		790 Refusal	Jar	Progressively finer w/depth indicating offlap or
											regression of lake level w/ time.
				82.9	83.5	-	-	Fishtail	-	-	Cleanout
40				83.5	84.0	83.5	84.7	3" Hvorslev	350	Steel Tube	Drive 1.3, rec. 1.3
				84.0	84.5				680		Soft gray uniform silt (ML). Gradational
40A				84.5	84.8	84.7	84.8		800 (refusal)	Jar	change from very fine sand above.
				84.8	85.0			Fishtail			Cleanout
41				85.0	85.5	85.0	86.8	3" Hvorslev	250	Tube	Drive 1.8' - SPL 1.9
				85.5	86.0				480		Soft gray uniform silt (ML) or very fine
41A				86.0	86.5	86.8	86.9		700	Jar	sand (SP).
				86.5	86.8				810		

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	lbs		CLASSIFICATION AND REMARKS
			87.7	86.9	87.7			~~3" Hvorslev~~			~~Drive 0.2, refused~~
42		87.7		87.7	87.9	87.7	87.9	3" Hvorslev		Jar	Drive 0.2 refused, V. hd. Stratum is a
											Hd, ~~poorly~~ unsorted gravelly, clayey fine
											sand (SC), dk gray. Contact w/ sand above
											is abrubt.
43				87.9	90.3	87.9	88.9	5" Denison bbl	500 to 700	Tube	Rotating carbaloy bit (tube above teeth).
43A						88.9	89.0			Jar	Drive 2.4, recov. 2.2
44						89.0	90.0			Tube	V. hd gray ~~poorly~~ unsorted sandy, gravelly
44A						90.0	90.1			Jar	clay (CL) (glacial till). Angular frag-
											ments of hard igneous rock in sandy, clay
											matrix. Probably "hardpan" of other
											borings. Same unit as SC above, gradational
											contact. Tube samples very strong, brittle,
											in good condition.

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BORING LOG
FIELD DATA

Project STERLING HEIGHTS SEWER Site EDISON CORRIDOR Date 12 MAY 80
Location STATION 126+87 AND 27' EAST OF SEWER CENTER LINE Job No. 441-6210.10GR21
Drill Rig FAILING Inspector J. DUNBAR Operator B. HARRIED Surface El 608.5 (MSL) Boring No. WES-5-80

SAMPLE NUMBER	1980 DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	PRESS	CONT	CLASSIFICATION AND REMARKS
	5/12	0.0		0.0	37.0	—	—	6¼" FISHTAIL	—	—	CLEANOUT - DUMP RUNS TO
			28.5								APPROX. 28.5' WHERE COLOR
											OF DRILLING FLUID CHANGED
											FROM BLACK TO GREY.
	5/13			—	37.0	—	—	6¼" FISHTAIL	—	—	CLEANOUT
				—	—	—	—	DENISON BBL	—	—	OBSTRUCTION AT 9.0' — PULLED
				0.0	12.0	—	—	8¾" ROCK BIT	—	—	CLEANOUT
				1.0	10.0	—	—	8" CASING	—	—	STICKUP 1.2' DEMM 88
				10.0	37.0	—	—	6¼" FISHTAIL	—	—	CLEANOUT
1	5/13			37.0	39.4	37.6	38.5	DENISON BBL	200	TUBE	CLAY ~ GREY IN COLOR, MED. HARD, STIFF,
1A						38.5	39.6			JAR	WITH 5% GRAVEL (ROUNDED ~ ½" dia.)

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-5-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Press	Cont	CLASSIFICATION AND REMARKS
											and Low Plasticity (CL) - [illegible]
											Upper [illegible]
				39.4	40.0	—		FISHTAIL	—		CLEANOUT – KEEP LOSING DRILLING
											FLUID TO GARBAGE DUMP AND PLUGGIN
											FISHTAIL
				10.0	25.0	—		8" CASING	—		STICKUP 0.8' DEPTH: 24.2
				—	24.2	—		FISHTAIL	—		CLEANOUT – LOSING CIRCULATION
				24.2	31.0	—		8" CASING	—		DRILLED CASING TO MAKE ROOM
											FOR ANOTHER SECTION – WOULDN'T
											PUSH.
	5-14			25.0	35.0	—		8" CASING	—		STICKUP 0.8' DEPTH 34.2'

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-5-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	PRESS.	CONT.	CLASSIFICATION AND REMARKS
				—	46.0			6¼" FISHTAIL			CLEANOUT
				46.0	42.4			DENISON Bbl			NO SAMPLE. PIECE OF CAR TIRE
											BLOCKED OFF Bbl. SOME "JUNK"
											MATERIAL INDICATES CLAY.
				42.4	43.0			FISHTAIL			CLEANOUT
2	5/14	28.5		42.4	44.8	42.9	44.7	DENISON Bbl.	200	S.T.	
2A	5/14					44.7	44.8			JAR	(SC) SANDY CLAY — GREY IN COLOR,
											SOFT, MODERATELY STIFF, LOW PLAST.,
					(TOP 0.5' JUNK MATERIAL)						AND CONTAINS APPROX 5% V. FINE
											GRAVEL RUN 2.4, SPL. 1.8' — DRILL
											ACTION INDICATES SOFTER/CHANGING
											MATERIAL.
				44.8	45.0			FISHTAIL			CLEANOUT

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El 608.5 Boring No. WES-5-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	PRESS.	CONT.	CLASSIFICATION AND REMARKS
											(GM)
3A	5/14		45.0	45.0	47.4	45.5	45.6	DENISON Bbl	180	JAR	GRAVELLY SILT — GREY, SOFT, STIFF,
3	5/14	45.0				46.1	47.3			S.T.	LOW PLAST, AND CONTAINS GRAVEL
3B	5/14					47.3	47.4			JAR	(ROUNDED - 1/8 to 1/4" diam.) - REC 1.8'
											SPL 1.2' - TOP 0.5' CAME OUT
											DURING EXAMINATION! V. TOP OF
											SPL. WAS WASHOUT/FALL-IN.
				47.4	48.0	✓		FISHTAIL	✓		CLEANOUT
											(ML)
4	5/14			48.0	50.4	48.0	49.1	DENISON Bbl	250 to 400	ST	SILT - SAME AS ABOVE EXCEPT
4A	5/14					49.1	49.2			JAR	NO GRAVEL WAS OBSERVED.
				50.4	51.4	✓		FISHTAIL	✓		CLEANOUT
											(SP)
5			49.2	51.4	53.8	51.4	51.5	DENISON Bbl.	100 to 200	JAR	SAND — UNIFORM, V. FINE GRND, GREY,
		49.2									MODERATE/LITTLE COHESION - SPL
											WASHED - REC 0.3 FT

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______

Location ______ Job No. ______

Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-5-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	PRESS	CONT.	CLASSIFICATION AND REMARKS
				53.8	54.0			FISHTAIL			CLEANOUT.
											(CL)
6	5/14		54.6	53.7	54.2	53.7	54.8	3" Hvorslev	390	ST.	CLAY ~ DENSE, V. HARD, DENSE,
6A	5/14	54.0		54.2	54.7	54.8	54.9		520	JAR	WITH 5% FINE GRAVEL, AND
				54.7	54.9				800		LOW PLASTICITY - GRAVEL IS ROUNDED
				54.9	55.5			FISHTAIL			CLEANOUT.
				55.5	57.9	—	—	DENISON Bbl	300 to 500	—	LOST SPL. - HARD MATERIAL -
											CATCHERS BUSTED! SEE ABOVE
				55.5	58.9	—	57.7	DENISON Bbl	300		CLAY ~ SAME AS ABOVE
7	5/14					57.7	58.8		to	S.T	TOP 0.5' BELIEVE is Junk. Spl
7A	5/14					58.8	58.9		500	JAR	Should be regarded with caution.
				58.9	59.0			FISHTAIL			CLEANOUT

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. WES-5-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	PRESS	CONT.	CLASSIFICATION AND REMARKS
8A	5/15		59.0	59.0	61.3	59.0	59.1	DENISON Bbl.	300±	JAR	SAND (SP) - UNIFORM, FINE GRND,
8	5/15	59.0 ~~ASSUMED~~				59.1	61.2			ST	GREY IN COLOR, WITH LITTLE COHESS.
8B	5/15		~~63.5~~			61.2	61.3			JAR	— BOTTOM OF TUBE CONTAINS CLAY/
		60.5 ~~ASSUMED~~									WHICH IS DENSE, V. HARD, LOW PL
											AND INTERBEDDED SAND STRINGERS.
				61.3	62.0			FISHTAIL			CLEANOUT
											(ML)
9	5/15			62.0	64.4	62.0	63.5	DENISON Bbl	300±	ST	SILT - GREY IN COLOR, DENSE,
9A						63.5	63.6			JAR	MODERATELY HARD, & PLAST.
											Rec. 1.6'
				64.4	65.0			FISHTAIL			CLEANOUT
10	5/15		63.6	65.0	67.4	65.0	66.0	DENISON Bbl	200 to 350	ST	SAND (SP) - GREY IN COLOR, V. FINE
10A	5/15	63.6				66.0	66.1			JAR	GRND, UNIFORM, & LITTLE TO NO
											COHESSION.

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BORING LOG
FIELD DATA

Site ______ Date ______

Project ______ Job No. ______

Location ______ Boring No. WES-5-80

Drill Rig ______ Inspector ______ Operator ______ Surface El ______

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER	Press	Cont.	CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				67.4	68.0			FISHTAIL			CLEANOUT.
—	5/15			67.9	68.4	NONE		3" Hvorslev.	450	—	NO SPL (SANDY MATERIAL
				68.4	68.9				500		FELL OUT OF BOTTOM.)
				68.9	69.4				650		
				69.4	69.9				700		
				69.9	70.1				800		
				END							

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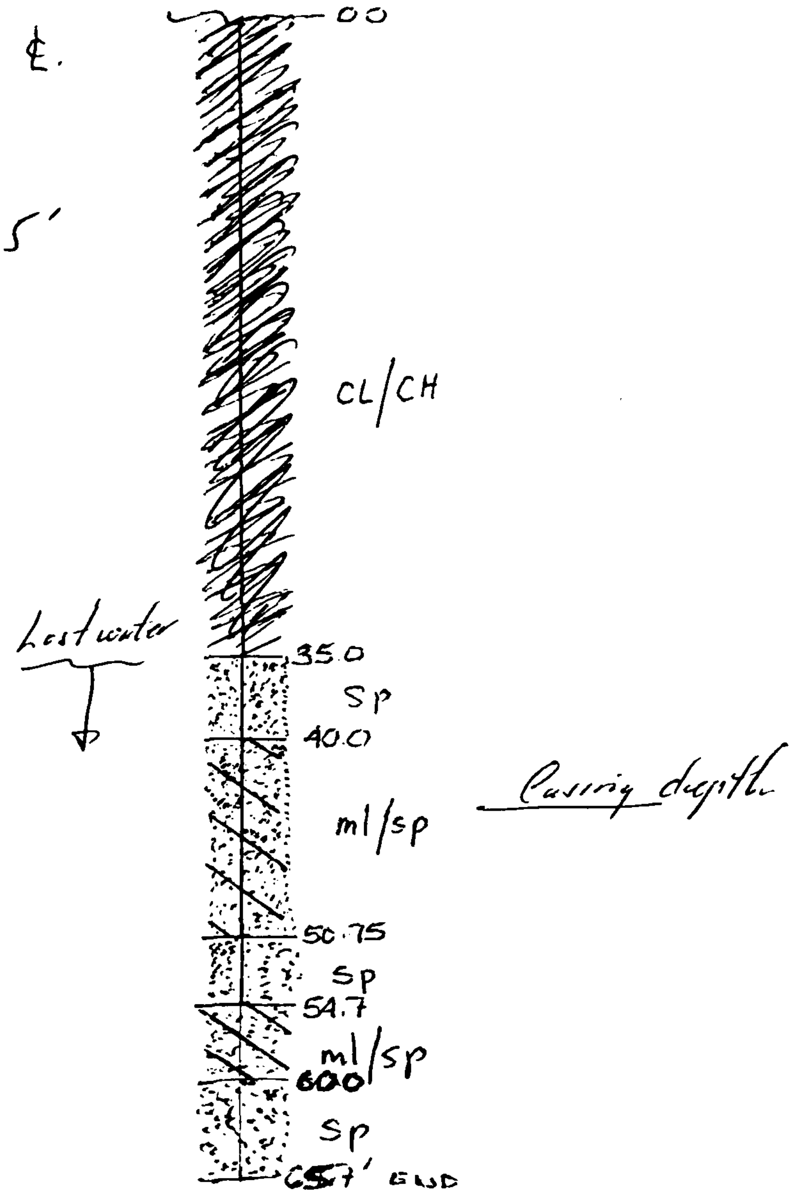
WES-9-80
6/21 to 7/2
114+25-12E ℄.

Casing to 46.5'



Filled with Sand to 35.0' & Grouted

BORING LOG
FIELD DATA

Project DETROIT SEWER FAILURE Site 15 Mile & Edison Corrid. Date 21 June 80
Location STATION 114+25 AND EAST OF ℄ 12.0' Job No. 441-E210.106K21
Drill Rig Failing Inspector Dunbar Operator B. HARRIED Surface El ____ Boring No. WES-9-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
	6/21	0.0	35±	0.0	36.8	—		5" Diam RkBit	—		CLEANOUT — LOST WATER AT
		ASSUMED									35.0' — MUDDED UP SEVERAL
											TIMES TO RETURN CIRCULATION.
				0.0	36.0	—		8¾" RkBit	—		CLEANOUT — 8" CASING.
	6/21			0.0	33.3	—		8" CASING	—		TOTAL: 34.5 STICKUP 1.2
	6/28			0.0	33.3	—		5" FISHTAIL	—		CLEANOUT — LOST WATER BELOW
											CS.
#1	6/28			0.0	45.0	—		7¾" Rk. Bit	—		CLEANOUT — RECOVERED SAMPLE
		35.0									OF CUTTINGS — SAND: Fine to
											med. GRND, UNKLSHN, i.e.
											Qtz., FeMg, Feldspars
				33.3	40.0	—		8" CASING	—		TOTAL 41.5' STICKUP 1.5'

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. LPS-9-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Cont.	CLASSIFICATION AND REMARKS
				—	44.0	—		7 3/4" Rk. Bit	—		CLEANOUT - LOST WATER BELOW CS.
				40.0	43.5	—		8" Casing	—		TOTAL 44.5' STICKUP 1.0'
			40.0	—	45.0	—		7 3/4" Rk Bit	—		Cleanout - Lost water below Cs.
#2	6/25	40.0 assumed		45.0	45.5	45.0	46.5	STANDARD SPLITSPOON	3	JAR	Silty Sand/Sand Silt - v. Fine grad,
				45.5	46.0				14		grey in color, fairly cohessive,
				46.0	46.5				10		and moderately dense, Sp/ml.
											Stratum depth is assumed.
				46.5	47.4	—		7 3/4" Rk Bit	—		CLEANOUT.
				47.4	49.4			STANDARD SPLITSPOON	✓		Silty Sand/Sandy Silt - see above.
				49.4	49.5				1		
				43.5	45.4	—		8" Casing	—		Total 46.5 Stickup 0.9'

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Cont	CLASSIFICATION AND REMARKS
				—	51.0	—		7¾" Rock Bit	—	—	CLEANOUT.
#3	6/28		50.75	51.0	51.5	51.0	52.5	STANDARD SPLITSPOON	4	SAR	Silty Sand – Fine grnd, grey
		50.75 Assumed		51.5	52.0				5		brown color, moderate cohesion,
				52.0	52.5				8		"hard", and contains approx
											5% v. fine gravel (¼ to ⅛"
											& angular sized fragments)
				52.5	56.8	—		7¾" Rk Bit	—		CLEANOUT – Last 1.8' is unaccounted.
											Material washed away. CLEANOUT
											went to 55.0 FT
	6/28		54.7	56.8	57.3	56.6	59.3	STANDARD SPLITSPOON	✓		
		54.7 Assumed		57.3	57.8				1		Silty Sand / Sandy Silt – Same as before
				57.8	59.3				4		

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

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BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector ______ Operator ______ Surface El ______ Boring No. ______

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blow	Cont.	CLASSIFICATION AND REMARKS
				59.3	60.0	✓	—	4½ Fishtail	1	—	Cleanout – Pump plugged/Blew check
#4	6/28	(SP)	60.0	60.0	60.5	60.0	61.5	STANDARD SPLITSPOON	2	JAR	Silty Sand – Fine grnd, moderate
				60.5	61.0				3		cohesion, grey in color, and
				61.0	61.5				11		approx 5% V. Fine gravel (same
											as before).
				61.5	62.0	—		4½" Fishtail	—	—	Cleanout.
#5	6/28	(SP)		62.0	62.5	62.0	63.5	STANDARD SPLITSPOON	18	JAR	Silty Sand – V. Fine grnd, uniform,
				62.5	63.0				30		grey, dense, & moderately
				63.0	63.5				51		cohesive.
				63.5	64.0	—		4½" Fishtail	1	—	Cleanout
#6	6/28	(SP)		64.0	64.5	64.0	64.7	STANDARD SPLITSPOON	36	JAR	Silty Sand – Same as above
				64.5	64.7				51		

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet 4 of 5 Sheets

BORING LOG
FIELD DATA

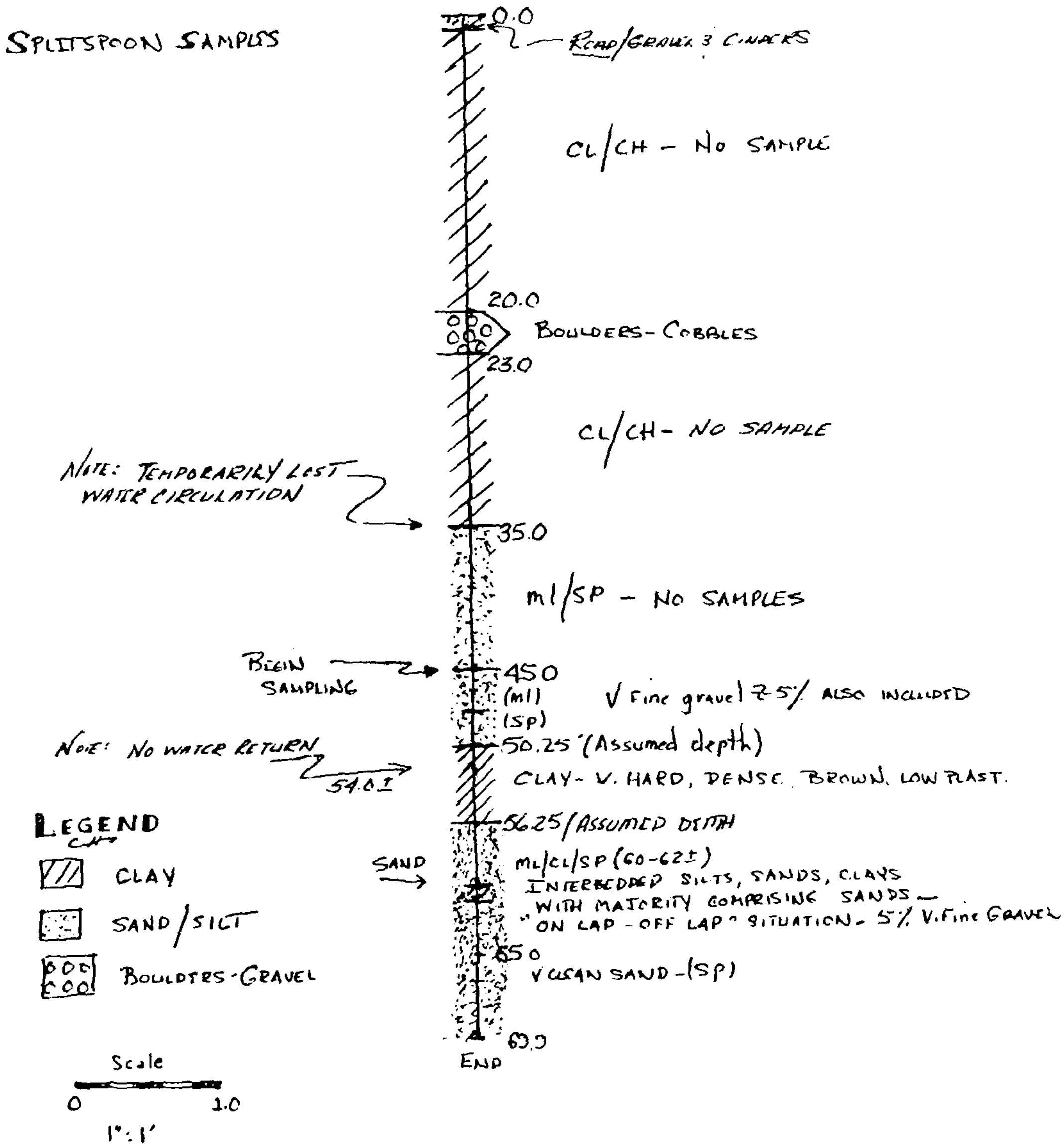
Project ____ Site ____ Date ____
Location ____ Job No. ____
Drill Rig ____ Inspector ____ Operator ____ Surface El ____ Boring No. WES-9-80

SAMPLE NUMBER	DATE TAKEN	STRATUM		DRIVE		SAMPLE		TYPE OF SAMPLER			CLASSIFICATION AND REMARKS
		FROM	TO	FROM	TO	FROM	TO				
				—	64.5	—	—	7 3/4" Rk Bit	—	—	Cleanout
#7	7/1			64.5	65.0	64.7	65.7	STANDARD SPLITSPOON	29	JHC	Silty Sand—Same (SP)
				65.0	65.5				40		
				65.5	65.7				51		
					End	of	Boring				
	2	Standard		Splitspoon		with		140lb	hammer		from 30" height

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet 5 of 5 Sheets

WES-10-80 — STATION 113+25 ; 13' WEST OF TUNNEL ¢
3 JULY TO 4 JULY 1980



STANDARD SPLITSPOON & 140 lbs HAMMER

BORING LOG
FIELD DATA

Project Sterling Heights Sewer Site 15 Mile & Edison Corid. Date 3 July 1980
Location Station 113+25 and 13 ft West of ℄ of Tunnel Job No. 441-6210.106R21
Drill Rig Failing Inspector J. Dunbar Operator B. Harred Surface El ______ Boring No. WES-10-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Container	CLASSIFICATION AND REMARKS
		0.0		0.0				8 3/4" Rock Bit	—	—	CLEANOUT ~ CLAY
											LARGE BOULDER(S) FROM 20.0± TO
			35±								23.0±'.
		35±									SAND/SILT ~ WATER LOST AND
					45.0						RETURNED AT 35±.
1	7/3	45±		45.0	45.5	45.0	46.5	STANDARD SPLITSPOON	17	JAR	SANDY SILT (ML) - GREY IN COLOR,
				45.5	46.0				30		MODERATE COHESSION, WITH
			47.25	46.0	46.5				23		MODERATE MOISTURE, AND 5%
											V. FINE GRAVEL, LOW PLASTICITY
				46.5	48.0	—	—	7 3/4" Rock Bit	—		CLEANOUT
2	7/3	47.25	(Assumed)	48.0	48.5	48.0	49.5	STANDARD SPLITSPOON	29	JAR	SILTY SAND (SP) GREY TO BROWN IN
				48.5	49.0				29		COLOR, MODERATE COHESSION, MOISTURE
			50.25	49.0	49.5				48		(MODERATE), FINE GRAINED, UNIFORM
											+ 5% V. FINE GRAVEL

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet 1 of 4 Sheets

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______

Location ______ Job No. ______

Drill Rig ______ Inspector J. Dunbar Operator B. Harried Surface El ______ Boring No. WES-10-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Cont	CLASSIFICATION AND REMARKS
				49.5	51.0	—	—	7 3/4" Rock Bit	—	—	CLEANOUT
3	7/3	50.25	(Assumed)	51.0	51.5	51.0	52.5	STANDARD SPLITSPOON	18	JAR	CLAY ~ BROWN IN COLOR, V. HARD,
				51.5	52.0				38		DENSE, WITH 5% V. FINE GRAVEL.
				52.0	52.5				30		LOW PLASTICITY, DRY
				52.5	54.0	—	—	7 3/4" Rock Bit	—	—	CLEANOUT
4	7/4			54.0	54.5	55.2	55.5	STANDARD SPLITSPOON	3	JAR	CLAY ~ SAME AS ABOVE: LOST
				54.5	55.0				6		WATER & FLUSHED TOP OF
			56.25	55.0	55.5				8		SPL AWAY IN CLEANOUT.
				55.5	57.0	—	—	7 3/4" Rock Bit	—	—	CLEANOUT
5	7/4	56.25	(Assumed)	57.0	57.5	57.0	58.5	STANDARD SPLITSPOON	7	JAR	SILTY SAND ~ V. FINE, UNIFORMED,
				57.5	58.0				14		GREY SAND, MODERATE COHESION,
				58.0	58.5				12		AND MODERATE MOISTURE.

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED Sheet 2 of 4 Sheets

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector DUNBAR Operator B. HARRED Surface El ______ Boring No. WES-10-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Cont.	CLASSIFICATION AND REMARKS
				585	60.0	—	—	7 3/4" Rock Bit	—	—	CLEANOUT.
6	7/4			60.0	60.5	600	61.5	STANDARD SPLITSPOON	8	JAR	CLAY + SAND ~ ON LAP-OFF LAP-
				60.5	61.0				30		ALTERNATING LAMINA OF CL & SP
				61.0	61.5				25		GREY BROWN IN COLOR ~ BREAKS
											ALL ALONG SANDY AREAS ~ Same as Above
											MAJORITY IS SAND.
				615	630	—	—	7 3/4" Rock Bit	—	—	CLEANOUT.
7	7/4			630	635	63.0	64.5	STANDARD SPLITSPOON	11		Silty SAND ~ SAME ~ 5% V. Fine
				63.5	64.0				25		GRAVEL
				64.0	64.5				26		
				64.5	66.0	—	—	7 3/4" Rock Bit	—	—	CLEANOUT
8	7/4			66.0	66.5	66.0	67.5	STANDARD SPLITSPOON	7	JAR	Silty Sand ~ V. Fine GRND, CLEAN
				66.5	67.0				17		UNIFORM, DENSE, SAND, MOD. COHESION

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet 3 of 4 Sheets

BORING LOG
FIELD DATA

Project ______ Site ______ Date ______
Location ______ Job No. ______
Drill Rig ______ Inspector J. DUNBAR Operator B. HARRIED Surface El ______ Boring No. WES-10-80

SAMPLE NUMBER	DATE TAKEN	STRATUM FROM	STRATUM TO	DRIVE FROM	DRIVE TO	SAMPLE FROM	SAMPLE TO	TYPE OF SAMPLER	Blows	Cont.	CLASSIFICATION AND REMARKS
				67.0	67.5				37		
				67.5	69.0	—	—	7 3/4 Rock Bit	—	—	CLEANOUT
9	7/4			69.0	69.5	69.0	69.9	STANDARD SPLITSPOON	31	JAR	Silty Sand - V. FINE GRN,
				69.5	69.9				50		UNIFORM CLEAN SAND.
				END.							
			STANDARD SPLITSPOON WITH 140 lb. HAMMER FROM								
			30" HEIGHT.								

WES FORM JAN 74 819 EDITION OF NOV 1971 MAY BE USED

Sheet 4 of 4 Sheets

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION	TYPE	LETTER SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS — GRAVELS	CLEAN GRAVEL	GW	GRAVEL, Well Graded, gravel-sand mixtures, little or no fines
		GP	GRAVEL, Poorly Graded, gravel-sand mixtures, little or no fines
	GRAVEL WITH FINES	GM	SILTY GRAVEL, gravel-sand-silt mixtures
		GC	CLAYEY GRAVEL, gravel-sand-clay mixtures
COARSE GRAINED SOILS — SANDS	CLEAN SAND	SW	SAND, Well Graded, gravelly sands
		SP	SAND, Poorly Graded, gravelly sands
	SANDS WITH FINES	SM	SILTY SAND, sand-silt mixtures
		SC	CLAYEY SAND, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS	ML	SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity
		CL	LEAN CLAY, Sandy Clay, Silty Clay, of low to medium plasticity
		OL	ORGANIC SILTS and organic silty clays of low plasticity
	SILTS AND CLAYS	MH	SILT, fine sandy or silty soil with high plasticity
		CH	FAT CLAY, inorganic clay of high plasticity
		OH	ORGANIC CLAYS of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		Pt	PEAT and other highly organic soils
WOOD		Wd	WOOD
SHELLS		Sl	SHELLS
NO SAMPLE			
		Sh	SHALE

NOTE: Soils possessing characteristics of two groups are designated by combinations of group symbols

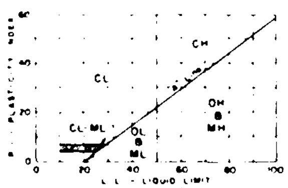
DESCRIPTIVE SYMBOLS

COLOR

COLOR	SYMBOL
TAN	T
YELLOW	Y
RED	R
BLACK	Bk
GRAY	Gr
LIGHT GRAY	lGr
DARK GRAY	dGr
BROWN	Br
LIGHT BROWN	lBr
DARK BROWN	dBr
BROWNISH GRAY	brGr
GRAYISH BROWN	grBr
GREENISH GRAY	gnGr
GRAYISH GREEN	grGn
GREEN	Gn
BLUE	Bl
BLUE GREEN	BlGn
WHITE	Wh
MOTTLED	Mot

CONSISTENCY FOR COHESIVE SOILS

CONSISTENCY	COHESION IN LBS/SQ FT FROM UNCONFINED COMPRESSION TEST	SYMBOL
VERY SOFT	< 250	vSo
SOFT	250 - 500	So
MEDIUM	500 - 1000	M
STIFF	1000 - 2000	St
VERY STIFF	2000 - 4000	vSt
HARD	> 4000	H



PLASTICITY CHART

For classification of fine-grained soils

MODIFICATIONS

MODIFICATION	SYMBOL
Traces	Tr
Fine	F
Medium	M
Coarse	C
Concretions	cc
Rootlets	rt
Lignite fragments	lg
Shale fragments	sh
Sandstone fragments	sds
Shell fragments	slf
Organic matter	O
Clay strata or lenses	CS
Silt strata or lenses	SIS
Sand strata or lenses	SS
Sandy	S
Gravelly	G
Boulders	B
Slickensides	SL
Wood	Wd
Oxidized	Ox
Crumbly	Cr
Loose	Lo
Vegetation	Veg

NOTES:

FIGURES TO LEFT OF BORING UNDER COLUMN "W OR D_{10}"

Are natural water contents in percent dry weight

When underlined denotes D_{10} size in mm *

FIGURES TO LEFT OF BORING UNDER COLUMNS "LL" AND "PL"

Are liquid and plastic limits, respectively

SYMBOLS TO LEFT OF BORING

- ▽ Ground-water surface and date observed
- (C) Denotes location of consolidation test **
- (S) Denotes location of consolidated-drained direct shear test **
- (R) Denotes location of consolidated-undrained triaxial compression test **
- (Q) Denotes location of unconsolidated-undrained triaxial compression test **
- (T) Denotes location of sample subjected to consolidation test and each of the above three types of shear tests **
- FW Denotes free water

FIGURES TO RIGHT OF BORING

Are values of cohesion in lbs/sq ft from unconfined compression tests

In parenthesis are driving resistances in blows per foot determined with a standard split spoon sampler (1⅜" ID, 2" OD) and a 140 lb driving hammer with a 30" drop

Where underlined with a solid line denotes laboratory permeability in centimeters per second of undisturbed sample

Where underlined with a dashed line denotes laboratory permeability in centimeters per second of sample remoulded to the estimated natural void ratio

* The D_{10} size of a soil is the grain diameter in millimeters of which 10% of the soil is finer, and 90% coarser than size D_{10}

** Results of these tests are available for inspection in the U.S. Army Engineer District Office, if these symbols appear beside the boring logs on the drawings

GENERAL NOTES:

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local variations characteristic of the subsurface materials of the region are anticipated and, if encountered, such variations will not be considered as differing materially within the purview of clause 4 of the contract.

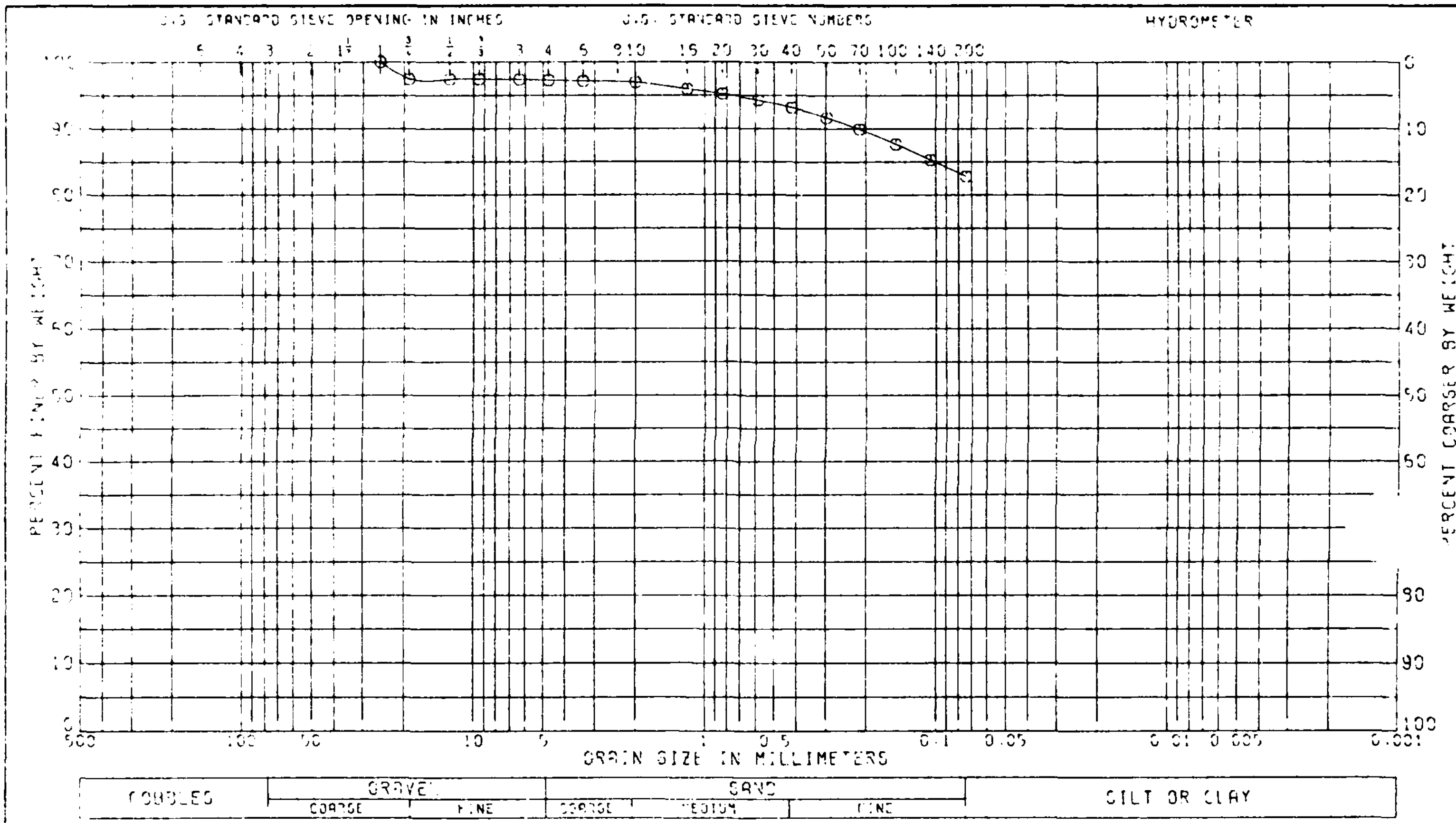
Ground water elevations shown on the boring logs represent ground water surfaces encountered on the dates shown. Absence of water surface data on certain borings implies that no ground water data is available, but does not necessarily mean that ground water will not be encountered at the locations or within the vertical reaches of these borings.

Consistency of cohesive soils shown on the boring logs is based on driller's log and visual examination and is approximate, except within those vertical reaches of the borings where shear strengths from unconfined compression tests are shown.

SOILS BORING LEGEND

APPENDIX B

GRADATION CURVES



LL 27	PL 15	PI 12	GS 2.74	NAT W.%	

CLASSIFICATION

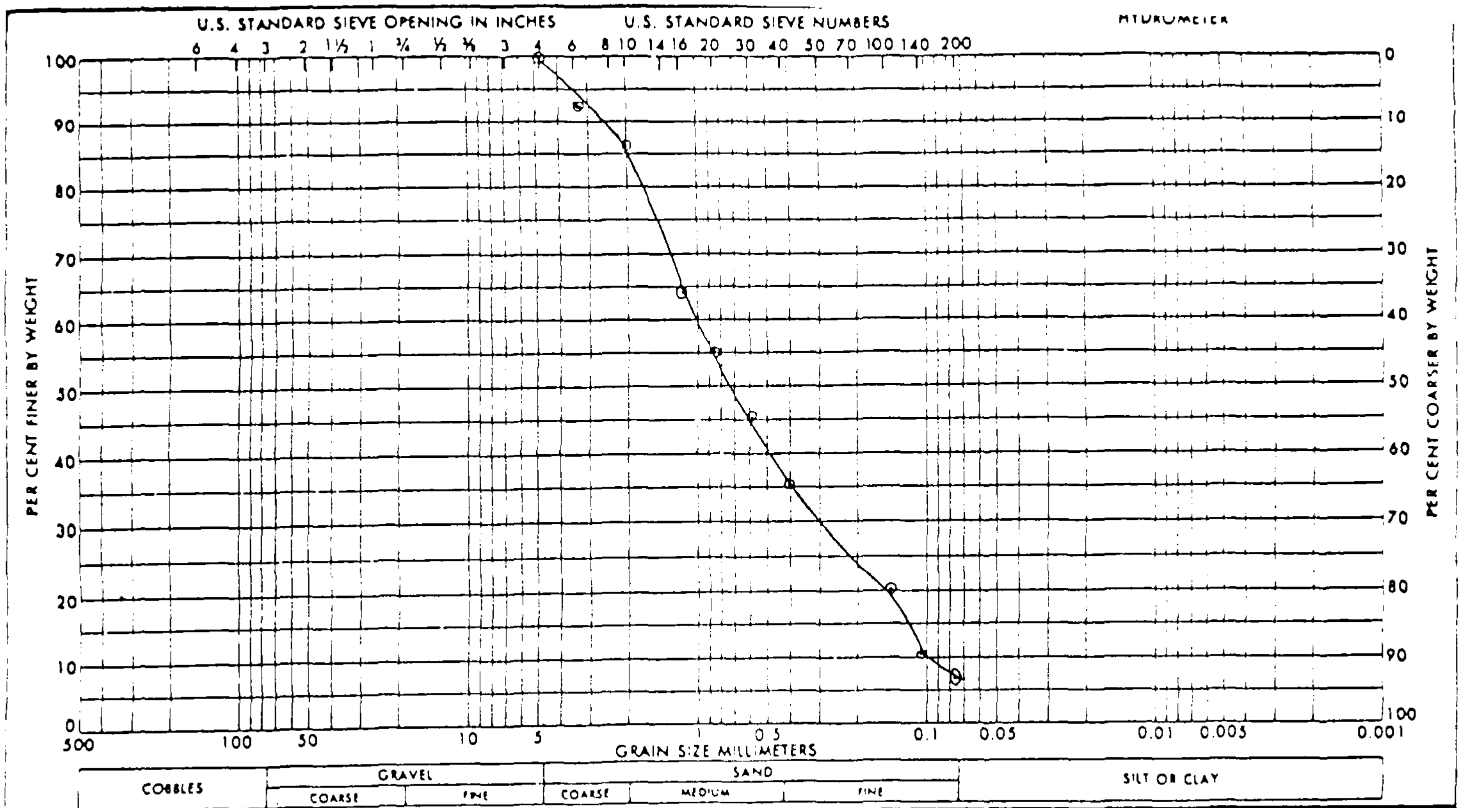
SANDY CLAY (CL), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO WES1 SAMPLE NO 32

DEPTH FEET 53.0/54.7 DATE 11 AUG 90



SAMPLE NO	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
34	59.5-61.1	SILTY SAND (SP-SM), BROWN		NP		

PROJECT DETROIT SEWER TUNNEL

AREA

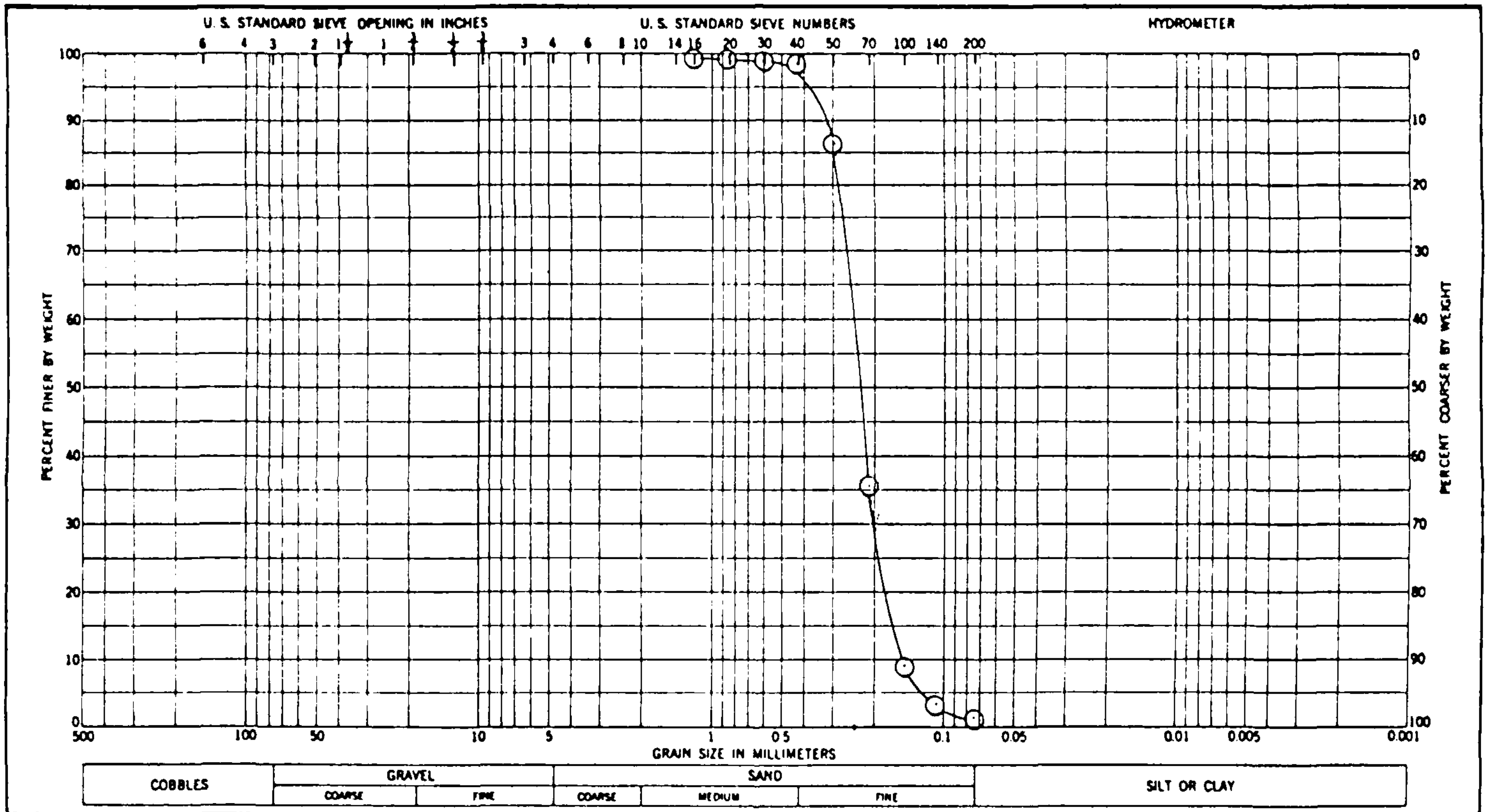
BORING NO WES 1

DATE 11 AUG 80

GRADATION CURVES

ENG FORM 1 MAY 63 2087 REPLACES WES FORM NO 1241, SEP 1962, WHICH IS OBSOLETE.

U.S. GOVERNMENT PRINTING OFFICE 1963 O—700-176



Sample No	Elev or Depth	Classification	Nat w %	LL	PL	PI
38.1	63.9-65.9	SAND (SP) GRAY				

Project DETROIT SEWER TUNNEL

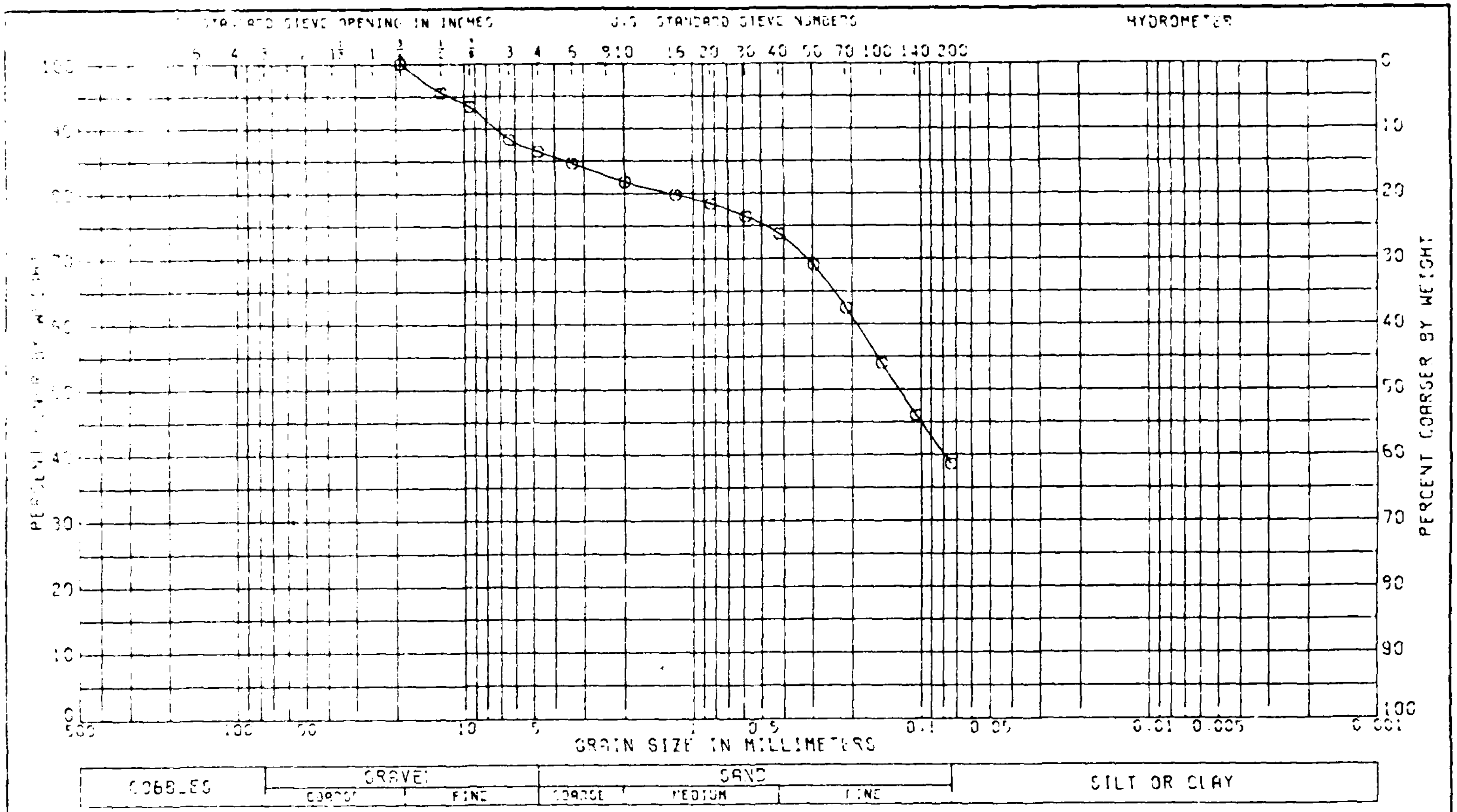
Area

Boring No. WES-1

Date 19 JUL 80

GRADATION CURVES

ENG FORM 1 MAY 63 2087



LL 20	PL 14	PI 6	GS	NAT W%	

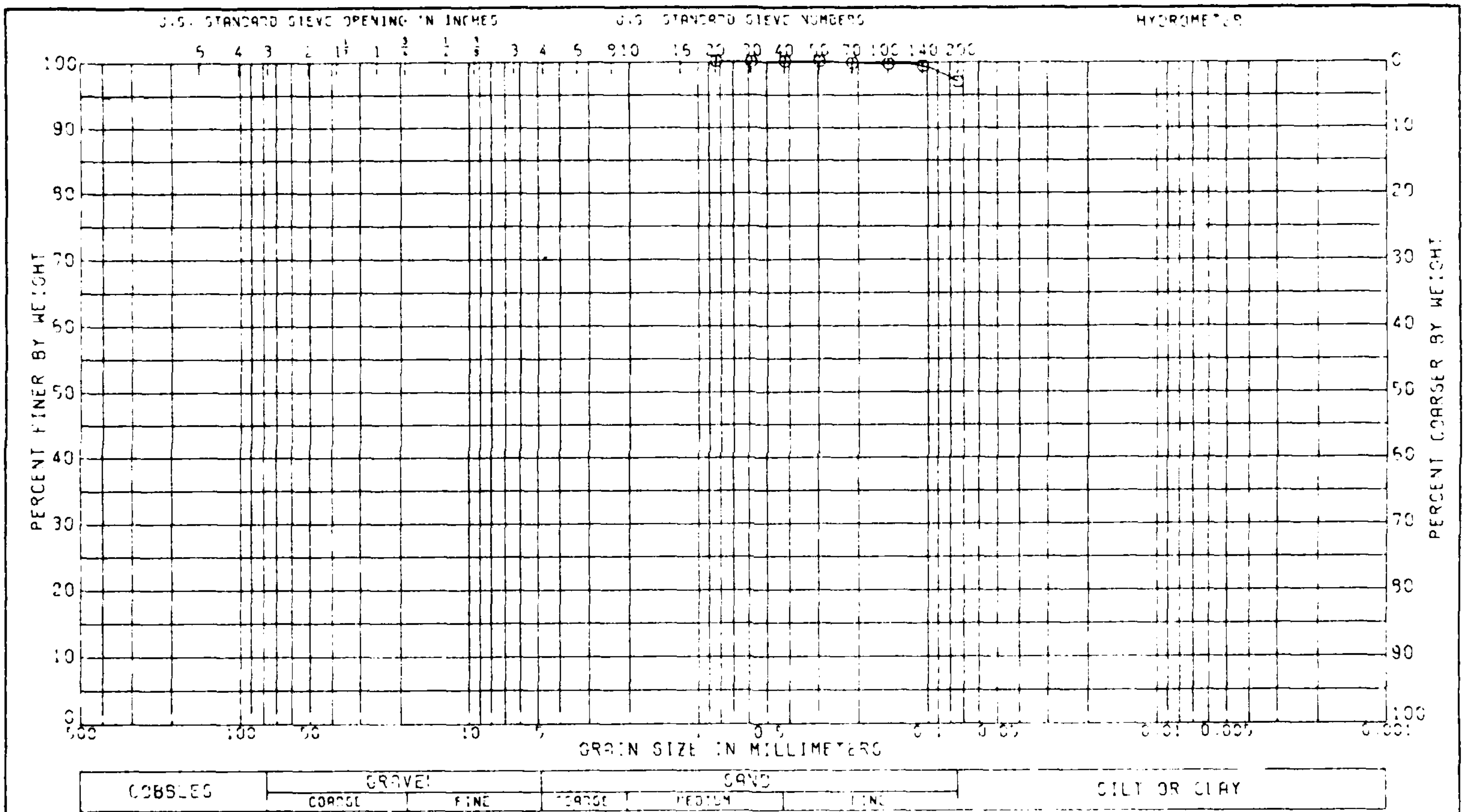
CLASSIFICATION
GRAVELLY SILTY SAND (SC-SM), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO WES3 SAMPLE NO. 15

DEPTH/ELEV 52.0/52.9 DATE 11 AUG 90



LL	PL	PI	GS	NAT W.%	

CLASSIFICATION

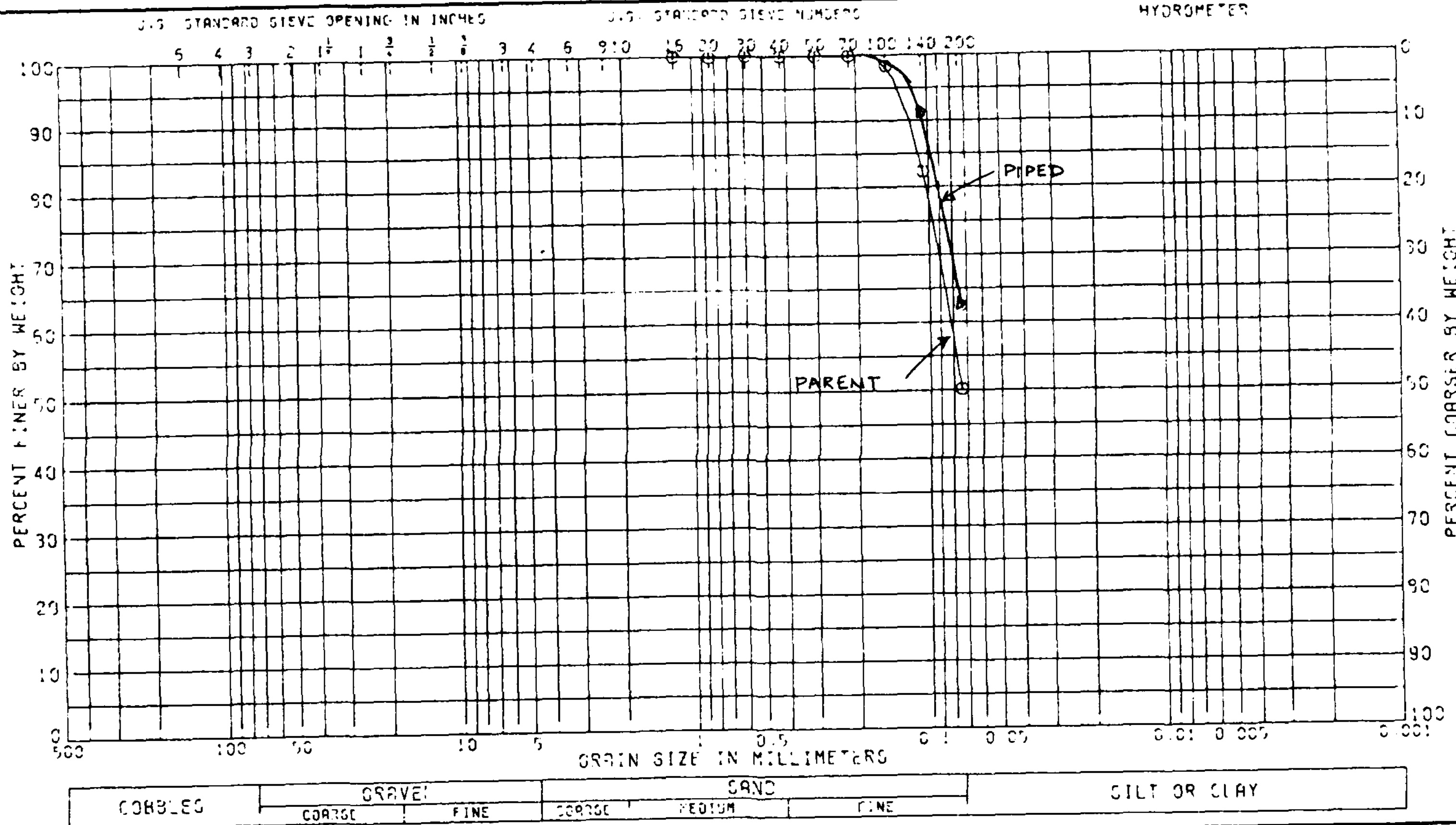
SILTY CLAY (CL), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO WS03 SAMPLE NO 23

DEPTH/ELEV 51.5/52.5 DATE 11 AUG 90



LL NP	PL	PI	GS	NAT W.%	

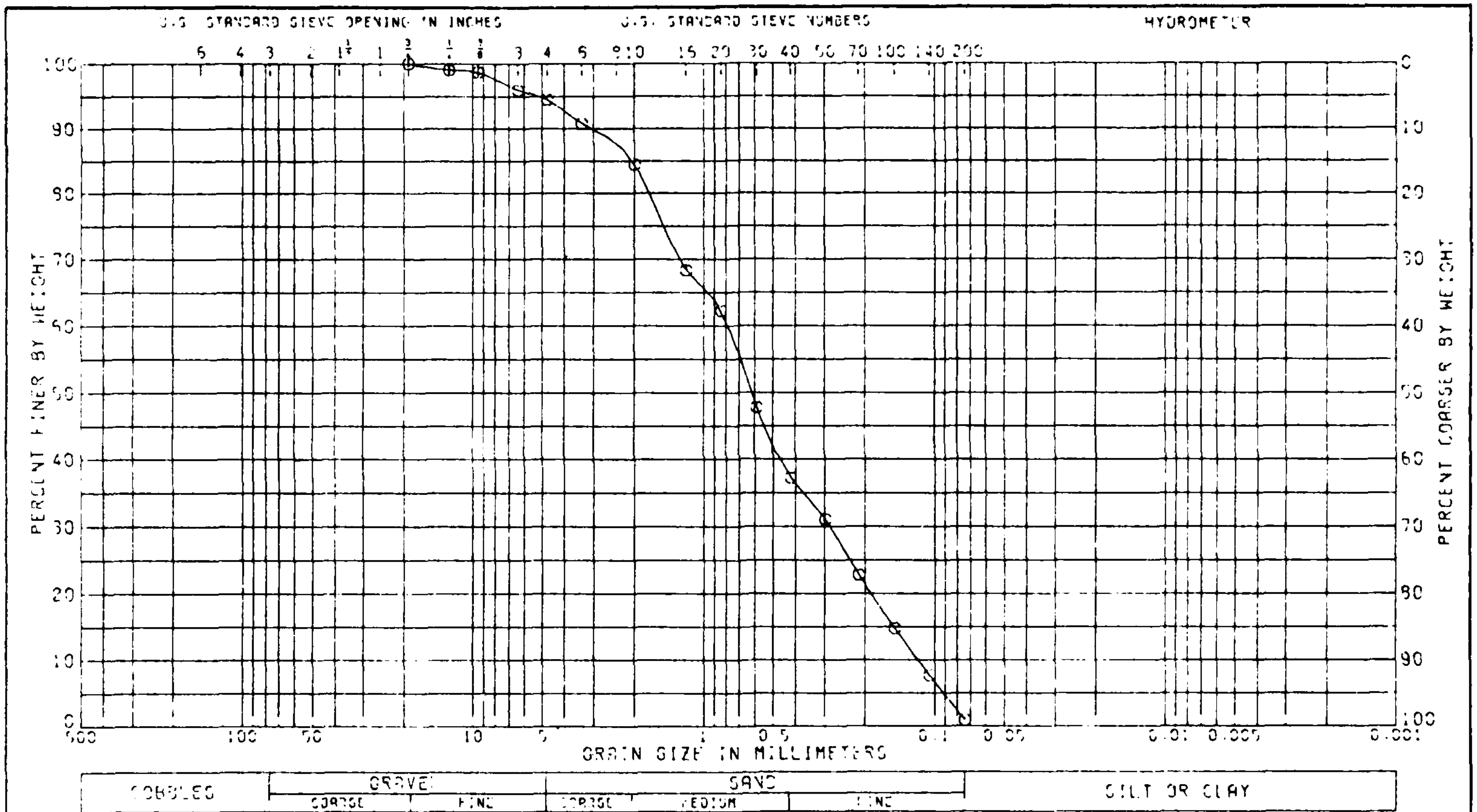
CLASSIFICATION
SANDY SILT (ML), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES3 SAMPLE NO. 31

DEPTH/ELEV 74.4/76.0 DATE 11 AUG 90



LL 14	PL 9	PI 5	CC	NAT W.%	

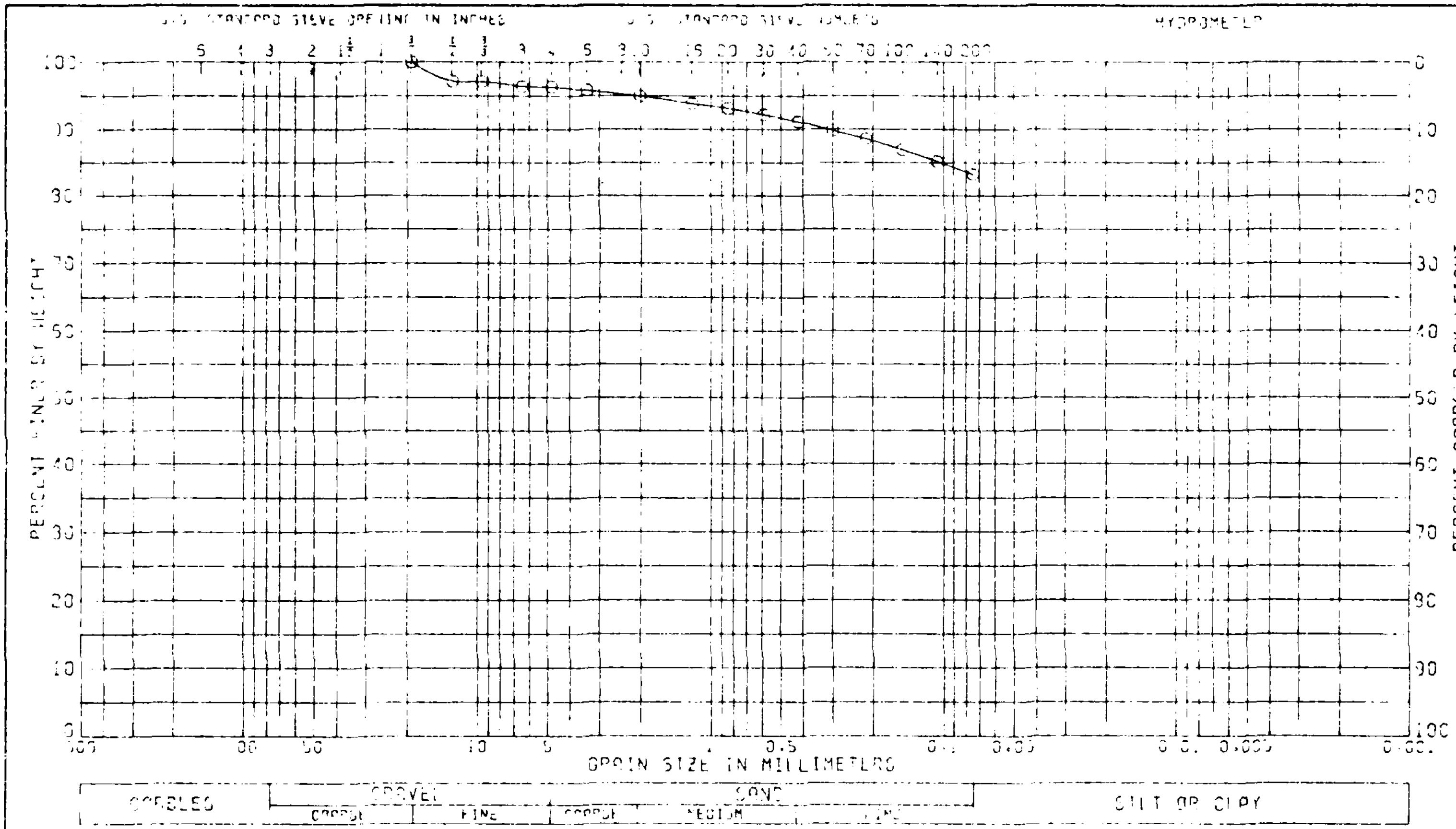
CLASSIFICATION
SAND (SP), GRAY W/GRAVEL

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES4 SAMPLE NO. 22

DEPTH/ELEV 17.5/49.2 DATE 11 AUG 90



LL 29	PL 12	PI 16	GS 2.74	NAT W%	

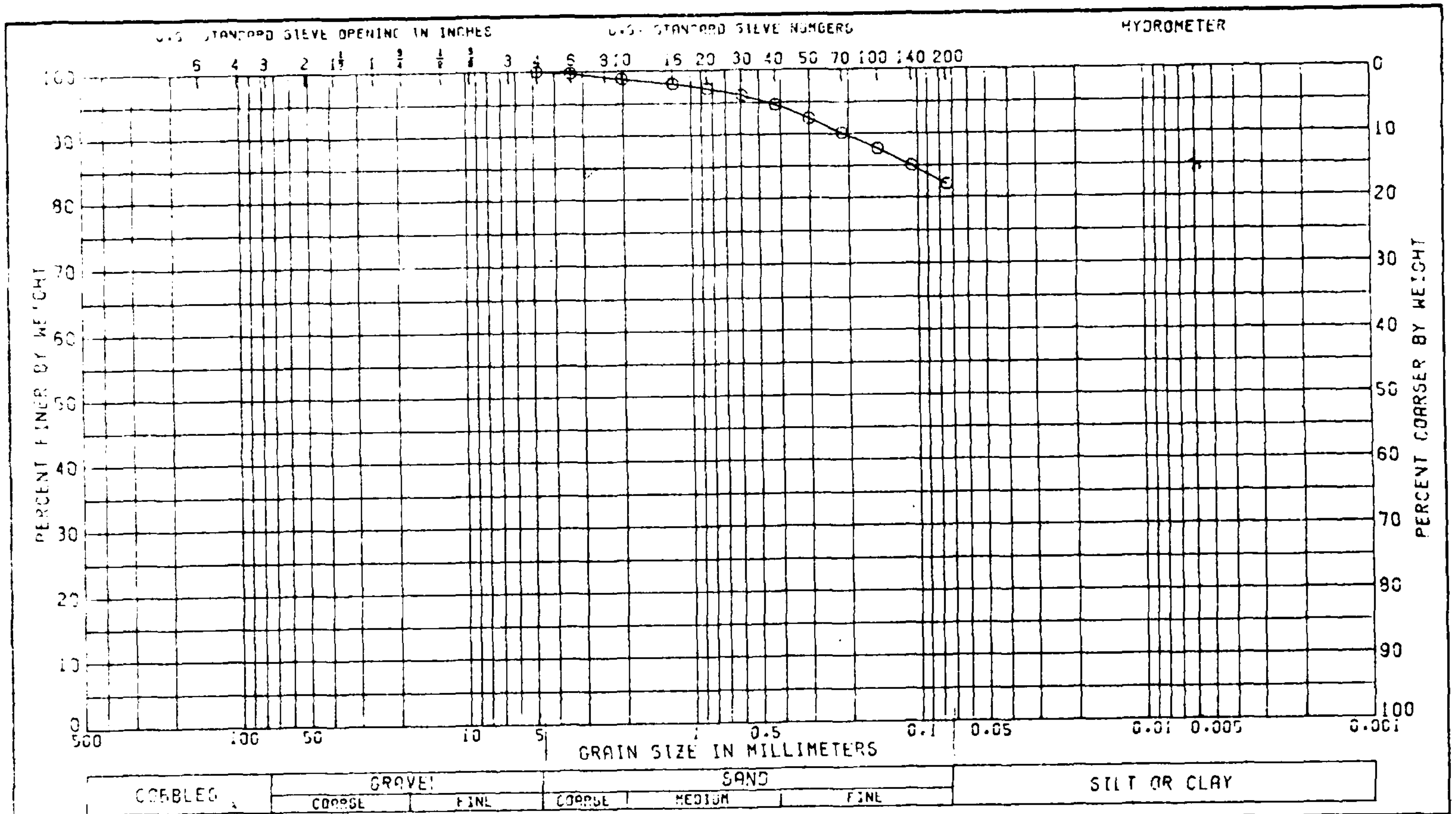
CLASSIFICATION
SANDY CLAY (CL) GRAY WITH TRACE GRAVEL

PROJECT DETROIT SEWER TUNNEL

BORING NO. REG-4 SAMPLE NO. 21

DEPTH/ELEV 50.0-51.1 DATE 01 JUL 80

GRADATION CURVE



LL 24	PL 12	PI 12	CG	NAT W.%	

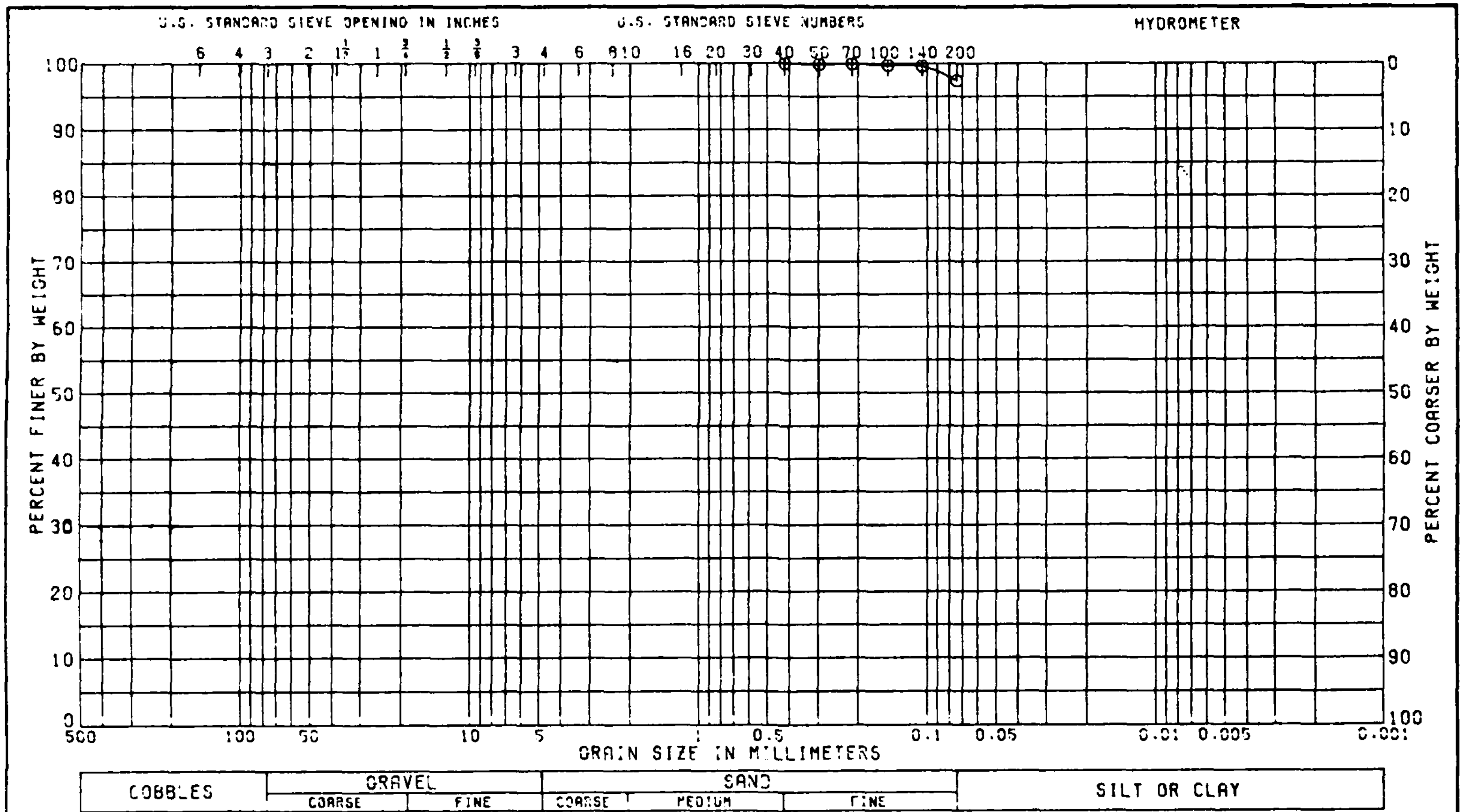
CLASSIFICATION
SANDY CLAY (CL), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES-4 SAMPLE NO 25

DEPTH/ELEV - 52.0-53.0 DATE 01 JUL 80



LL	PL	PI	GS	NAT W,%	

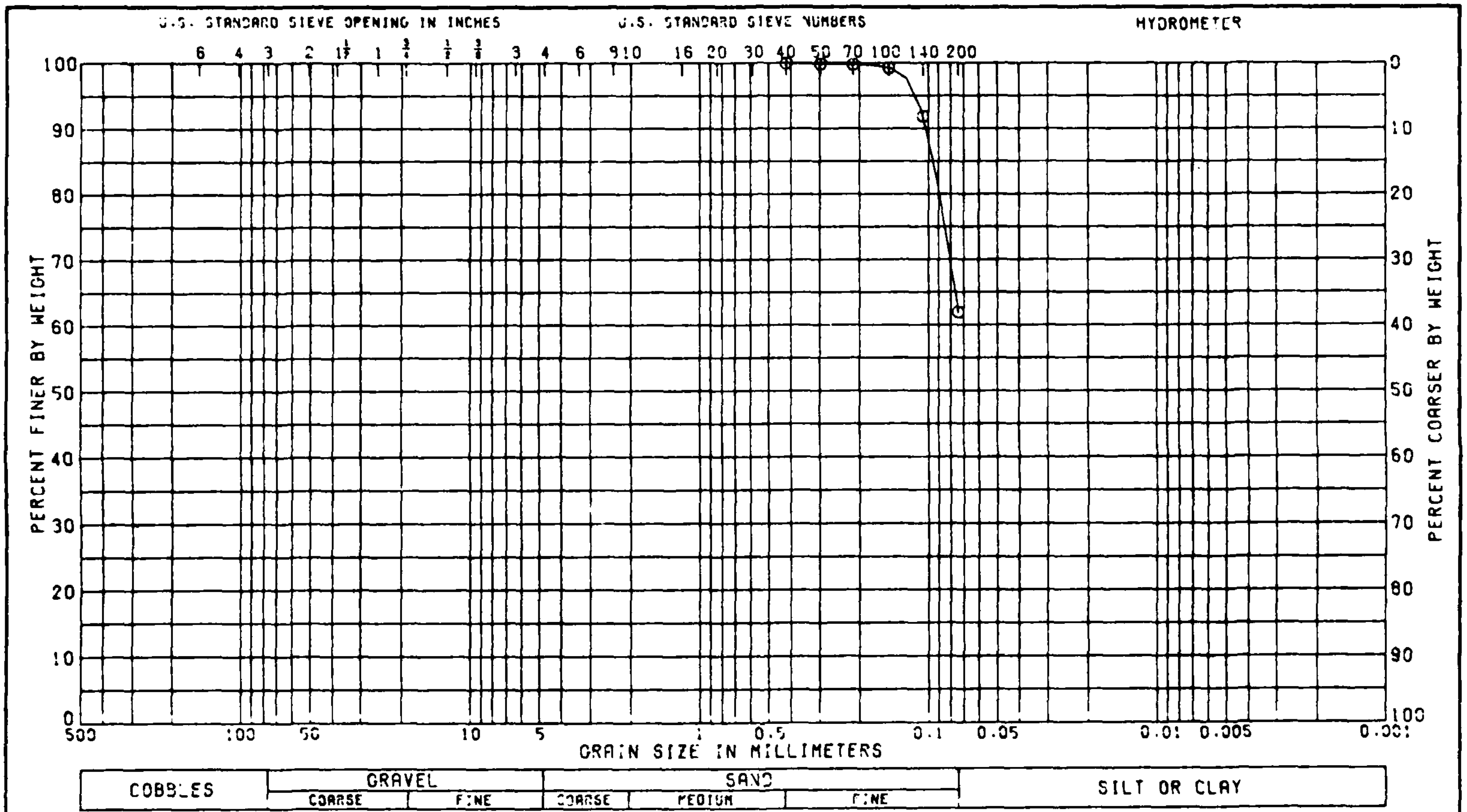
CLASSIFICATION
SILT (ML), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES4 SAMPLE NO. 30

DEPTH/ELEV 64.0/65.0 DATE 11 AUG 80



LL	PL	PI	GS	NAT W.%	

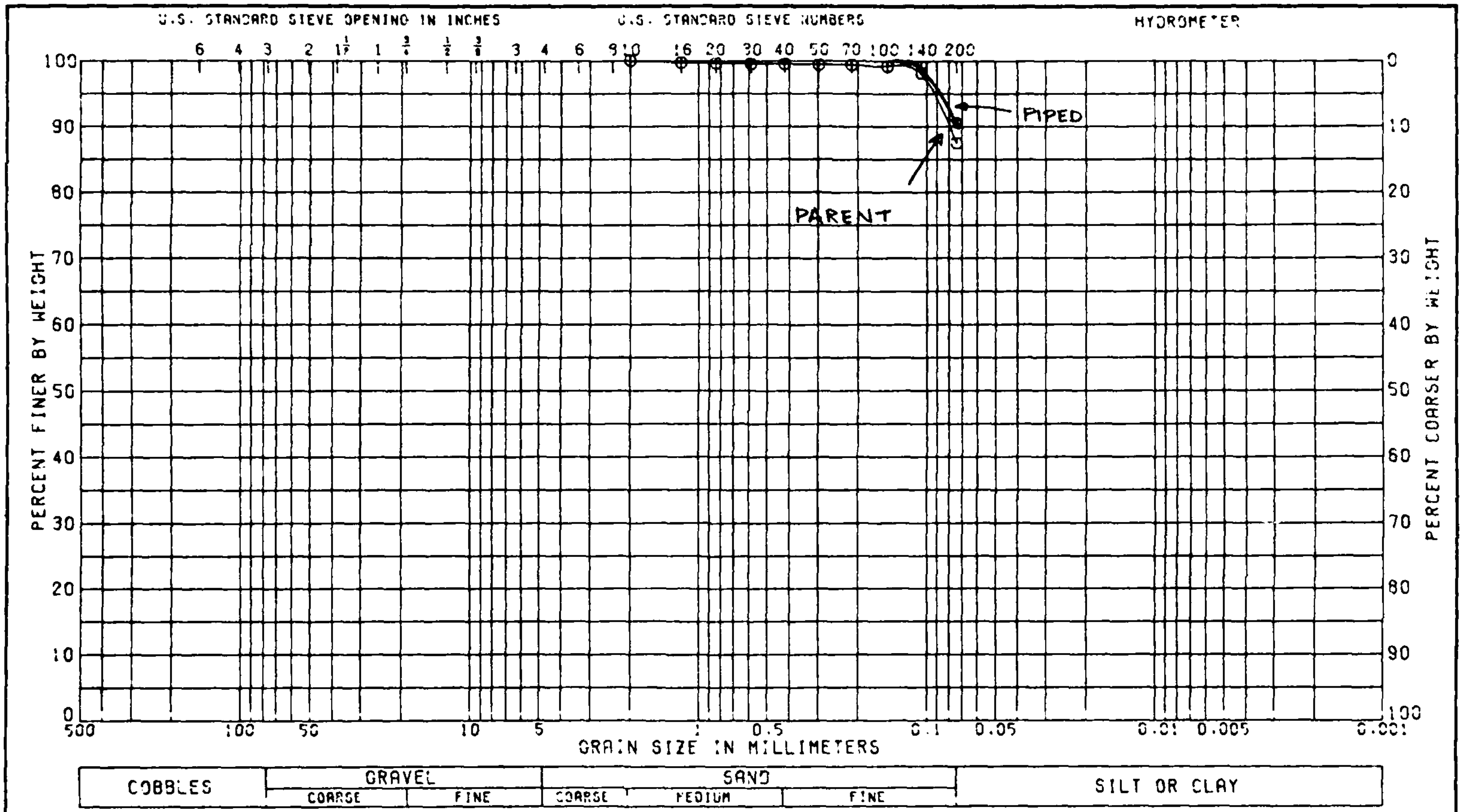
CLASSIFICATION
SANDY SILT (ML), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES4 SAMPLE NO. 31

DEPTH/ELEV 66.5/66.8 DATE 11 AUG 90



LL	PL	PI	GS	NAT W.%	

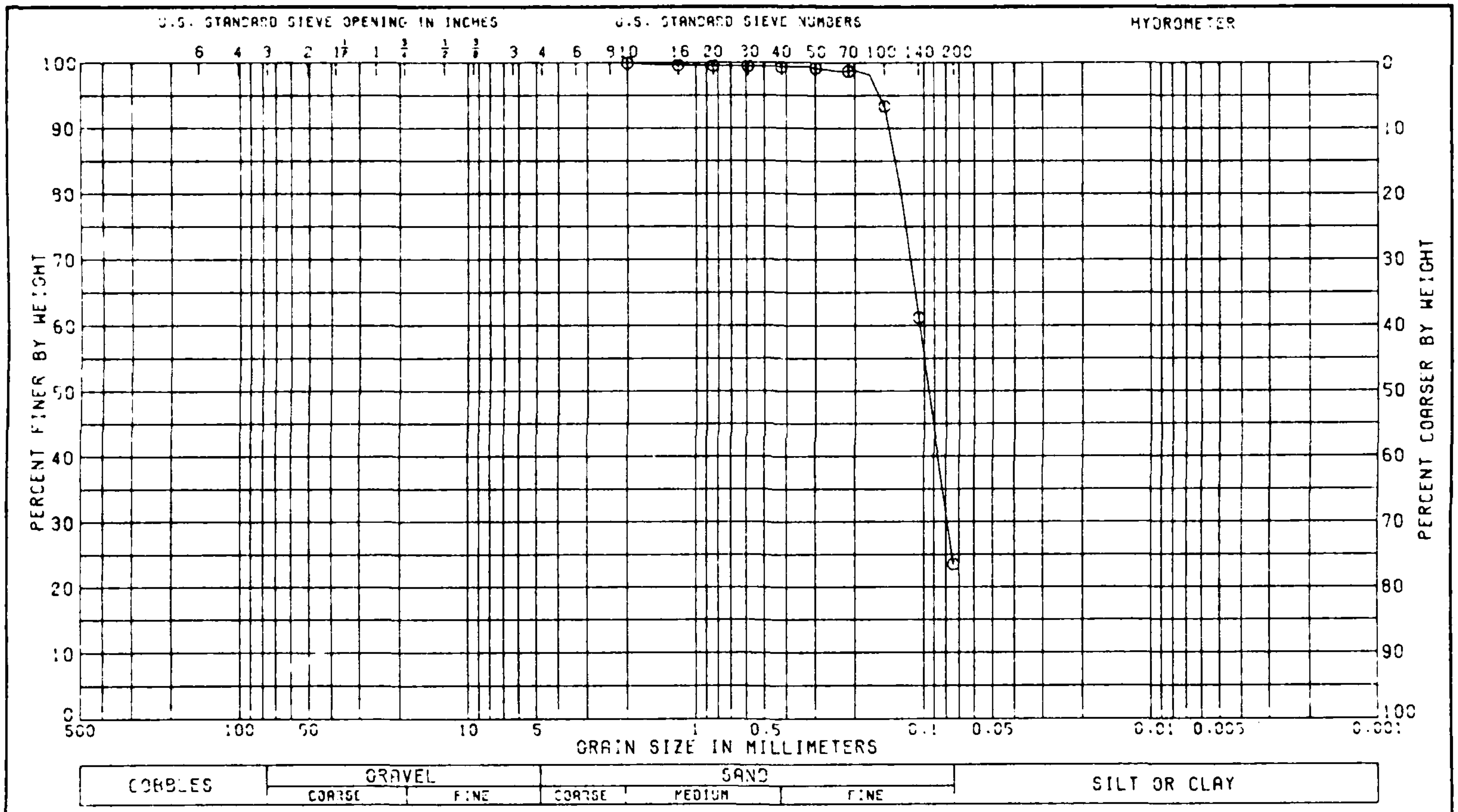
CLASSIFICATION

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. WES4 SAMPLE NO. 32

DEPTH/ELEV -69.0/70.0 DATE 11 AUG 90



LL NP	PL	PI	GS	NAT W.%	

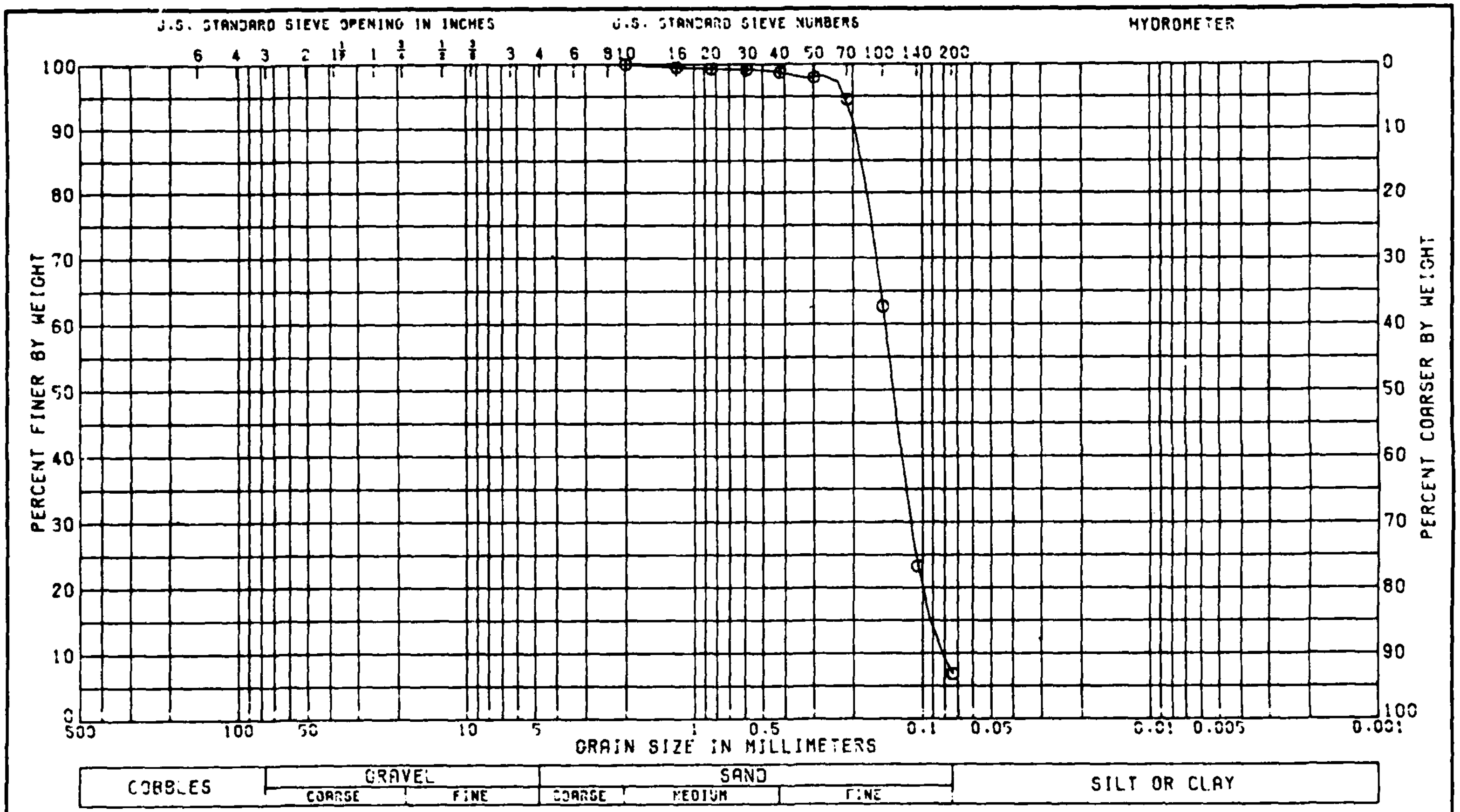
CLASSIFICATION SILTY SAND (SM), GRAY

GRADATION CURVE

PROJECT DETROIT SEWER TUNNEL

BORING NO. BREAK-1 SAMPLE NO. 1

DEPTH/ELEV 0 DATE 11 AUG 90



LL NP	PL	PI	GS	NAT W.%	

CLASSIFICATION

SILTY SAND (SP-SM), GRAY

GRADATION CURVE

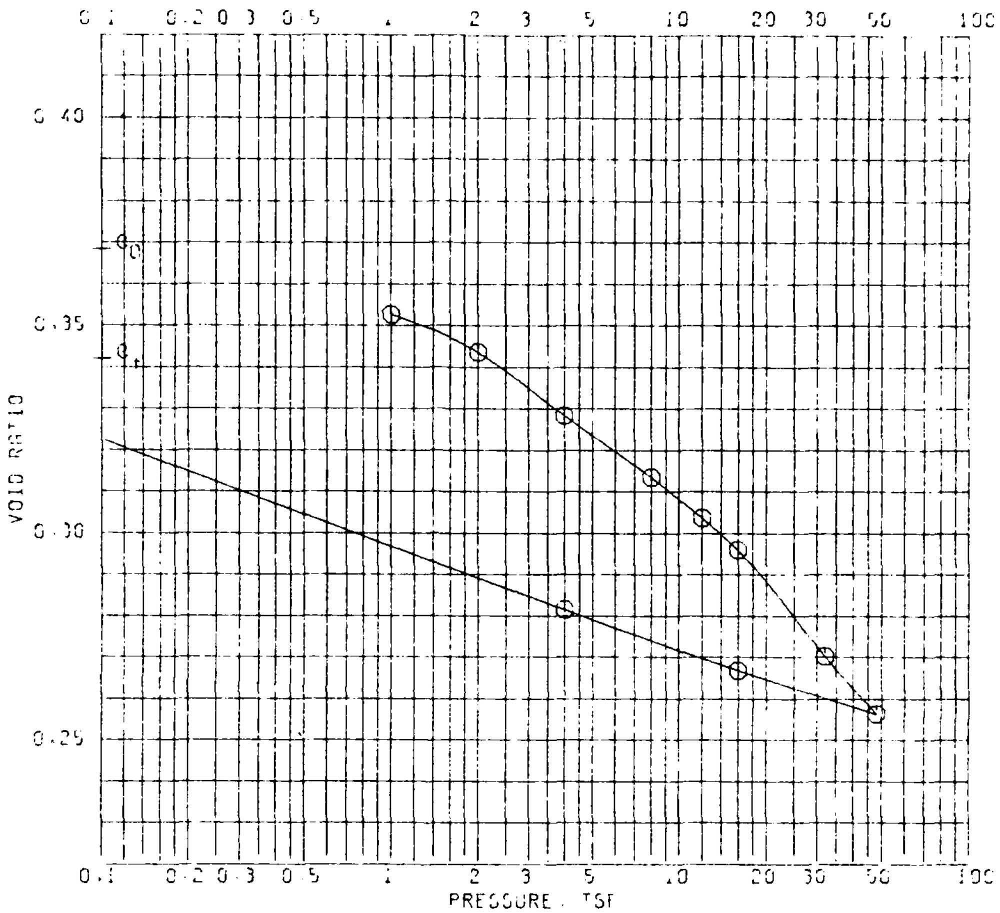
PROJECT DETROIT SEWER TUNNEL

BORING NO. BREAK-1 SAMPLE NO. 2

DEPTH/ELEV 0 DATE 11 AUG 80

APPENDIX C

CONSOLIDATION TEST DATA



			BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF		WATER CONTENT, %	12.4	12.3
PRECONSOL. PRESSURE, TSF		DRY DENSITY, PCF	125.0	127.5
COMPRESSION INDEX		SATURATION, %	92.6	100 +
TYPE SPECIMEN	UNDISTURBED	VOID RATIO	0.369	0.342
DIA. IN 4.44	HT. IN 1.121	BACK PRESSURE, TSF		

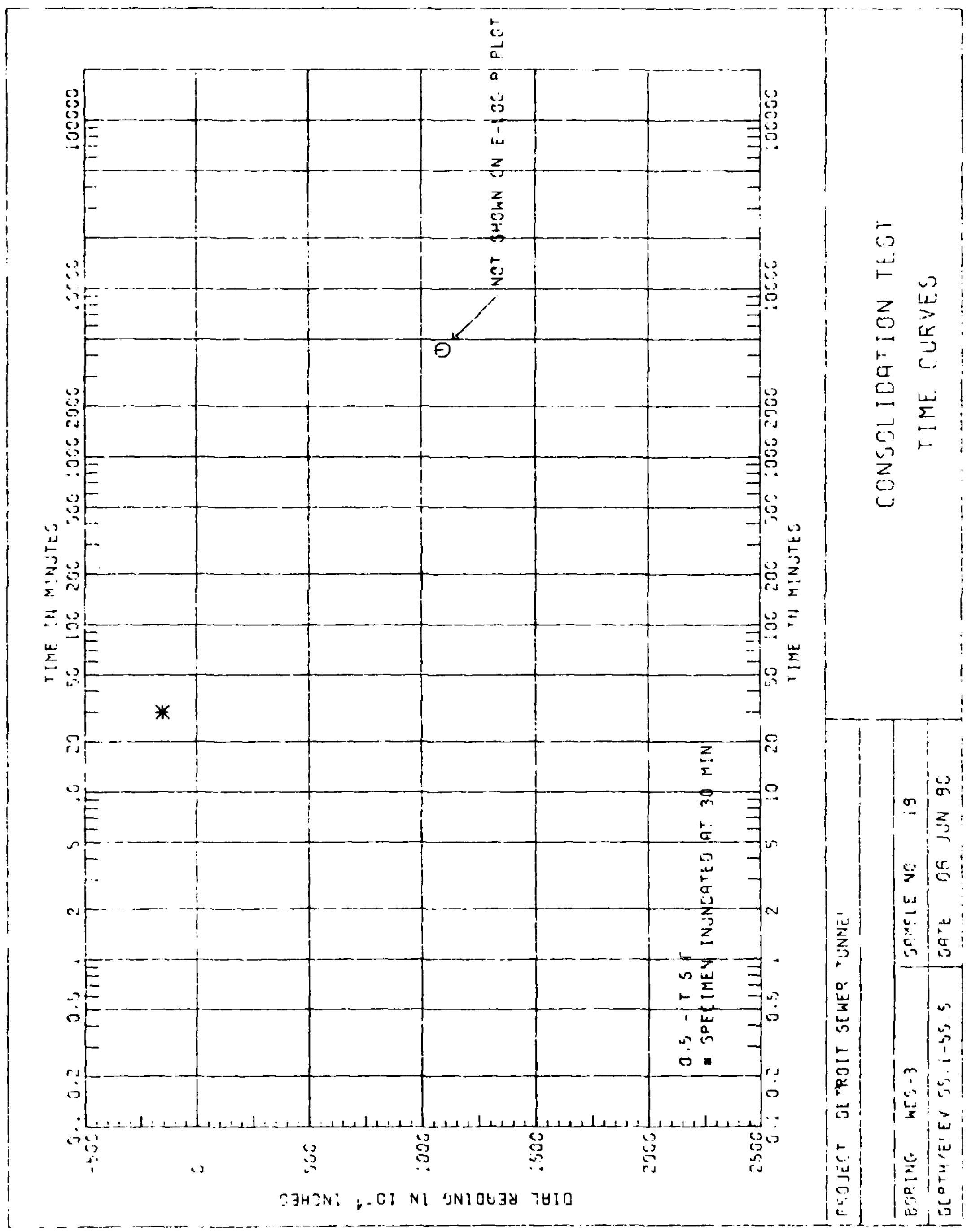
CLASSIFICATION CLAY (CL), BROWNISH GRAY, GRAVEL TO 1/4"		
LL 25	PL 13	PI 12
GS 2.74 COM	D_{10}	

PROJECT DETROIT SEWER TUNNEL

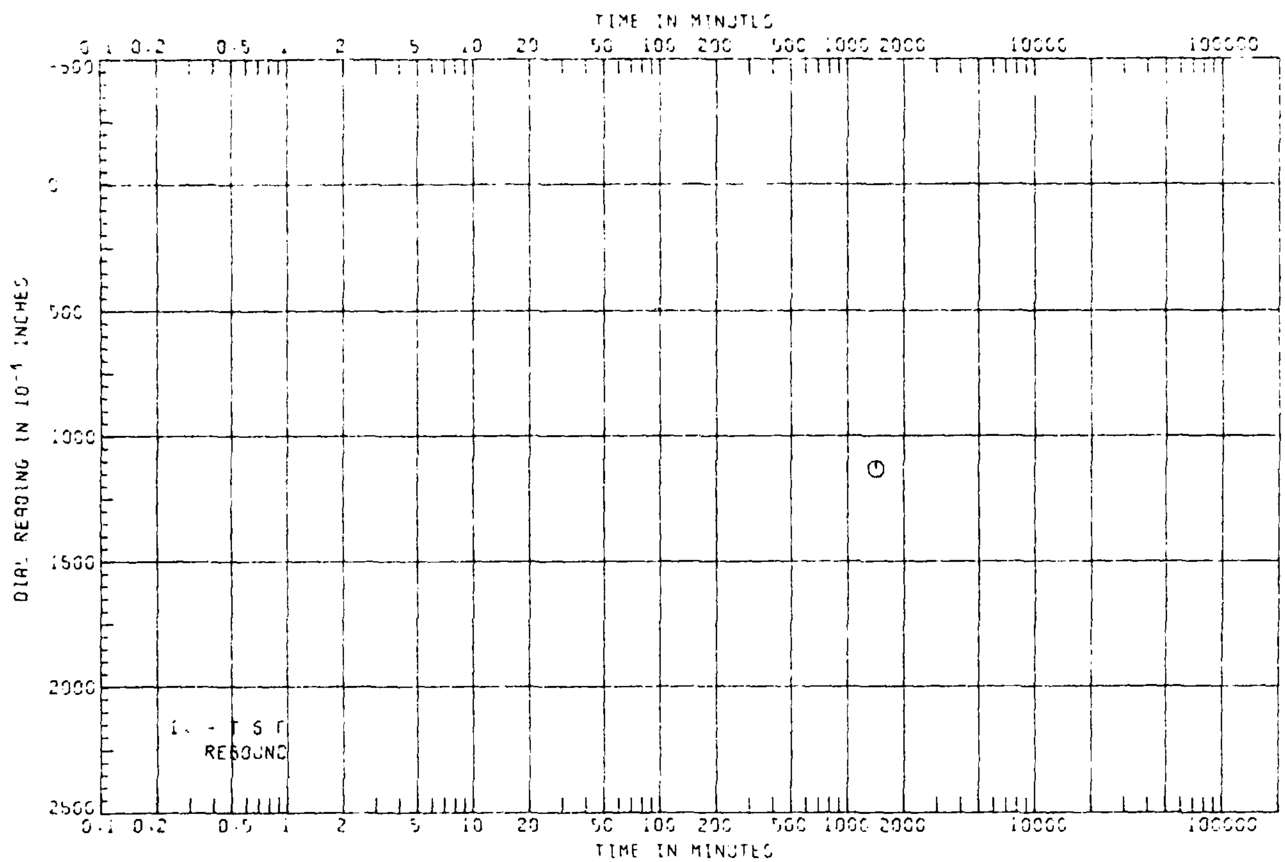
REMARKS	
BORING NO WES-3	SAMPLE NO 19
DEPTH/ELEV 55.0-55.5	DATE 06 JUN 90

CONSOLIDATION TEST REPORT

SHEET 1 OF 13



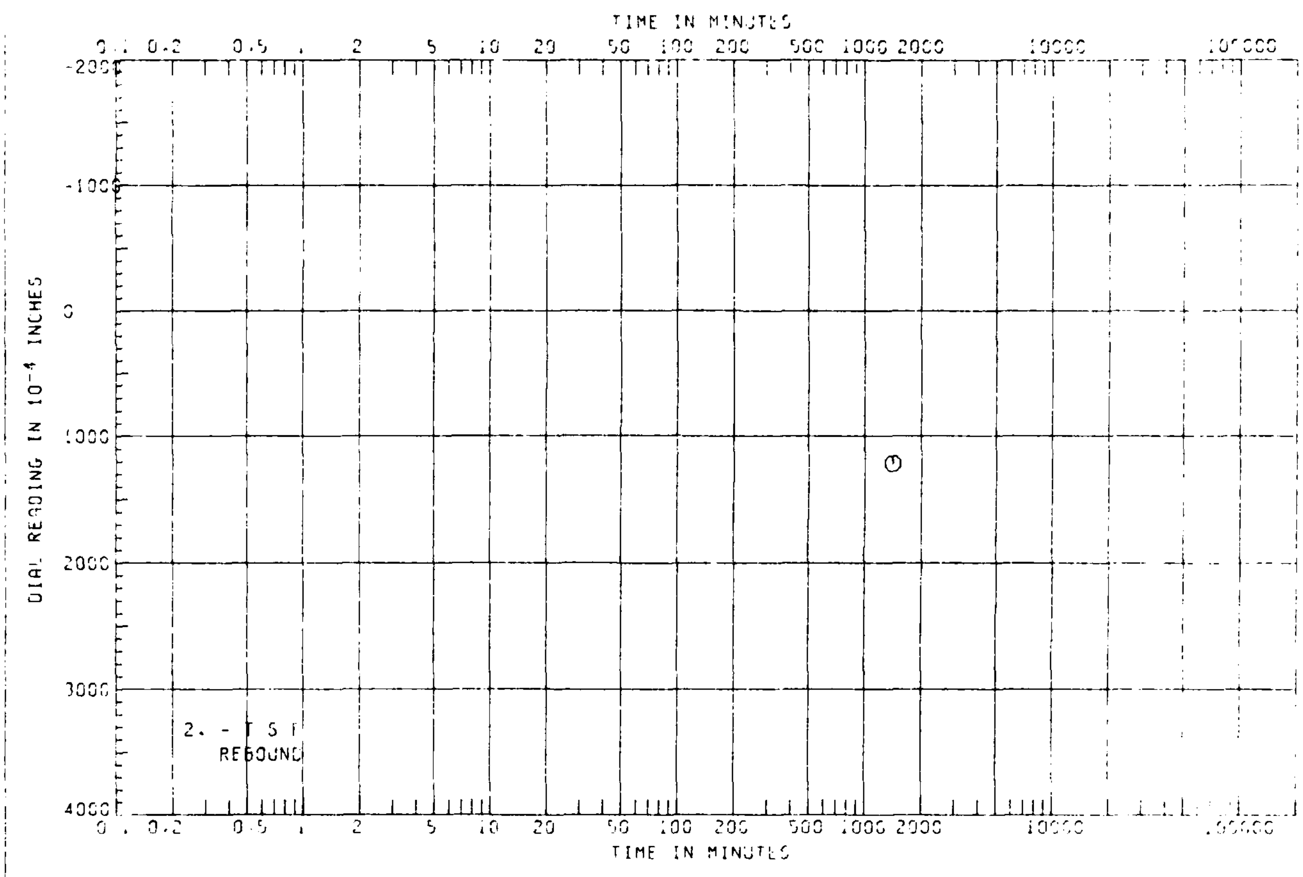
TIME IN MINUTES
DIAL READING IN 10^-4 INCHES
SPECIMEN INUNDATED AT 30 MIN
CONSOLIDATION TEST
TIME CURVES
PROJECT DETROIT SEWER TUNNEL
BORING WES-3
SAMPLE NO 19
DEPTH/ELEV 55.1-55.5
DATE 06 JUN 90



PROJECT DETROIT SEWER TUNNEL	
BORING WES-3	SAMPLE NO. 19
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 90

CONSOLIDATION TEST
TIME CURVES

SHEET 3 OF 13

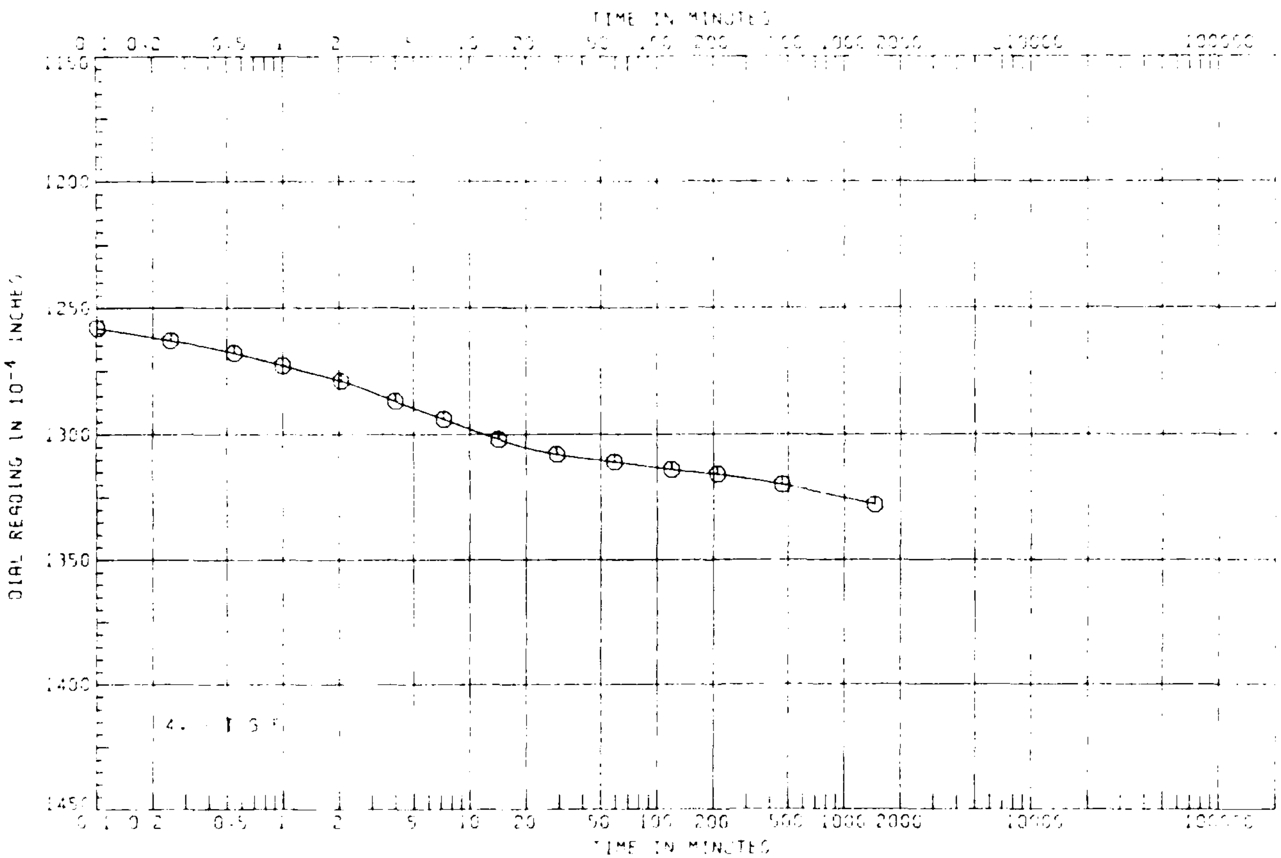


PROJECT DETROIT SEWER TUNNEL	
BORING WES-3	SAMPLE NO 19
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 90

SHEET 4 OF 13

CONSOLIDATION TEST

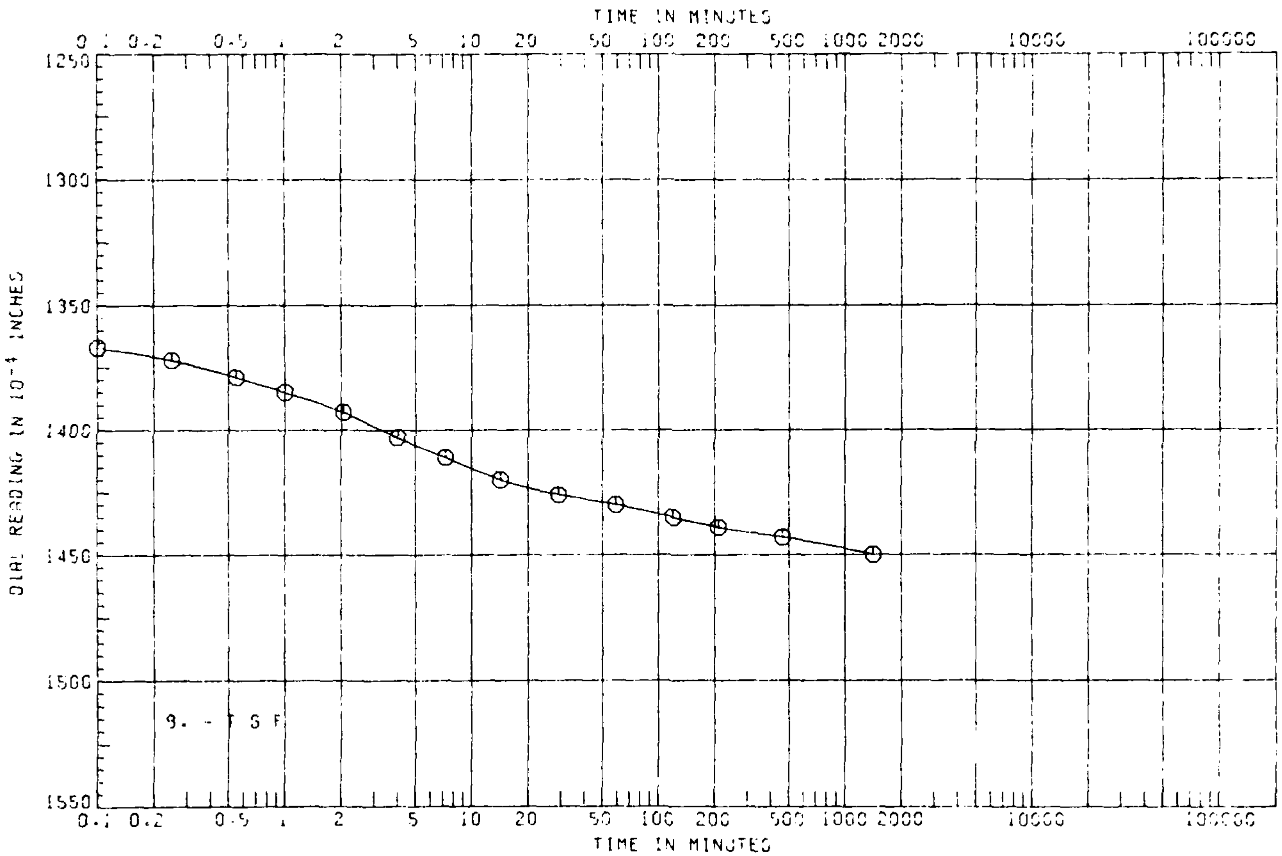
TIME CURVES



PROJECT DETROIT SEWER TUNNEL	
BORING WES-3	SAMPLE NO. 9
DEPTH/ELEV 55.1-55.5	DATE 05 JUN 90
SHEET 5 OF 13	

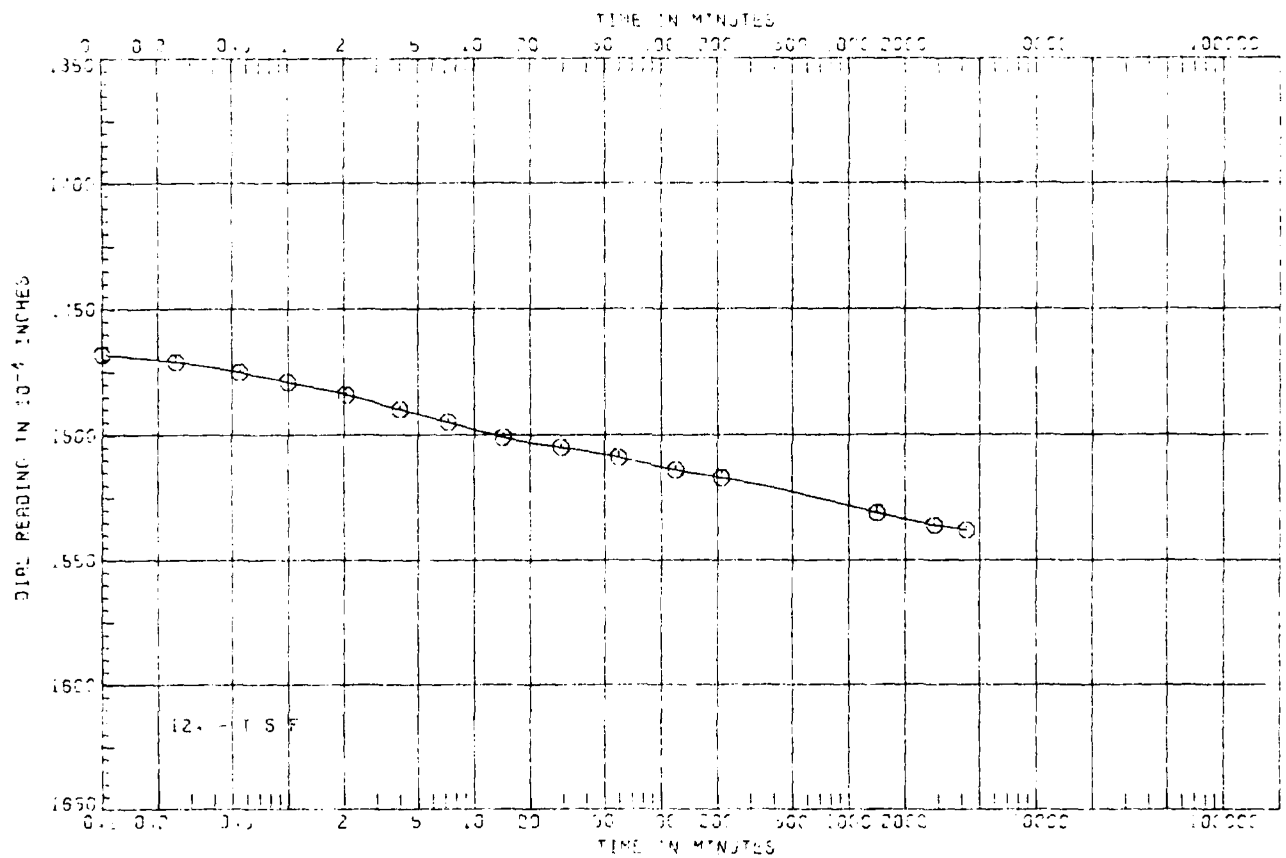
CONSOLIDATION TEST

TIME CURVES



PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-3	SAMPLE NO 19	
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 80	

SHEET 5 OF 13



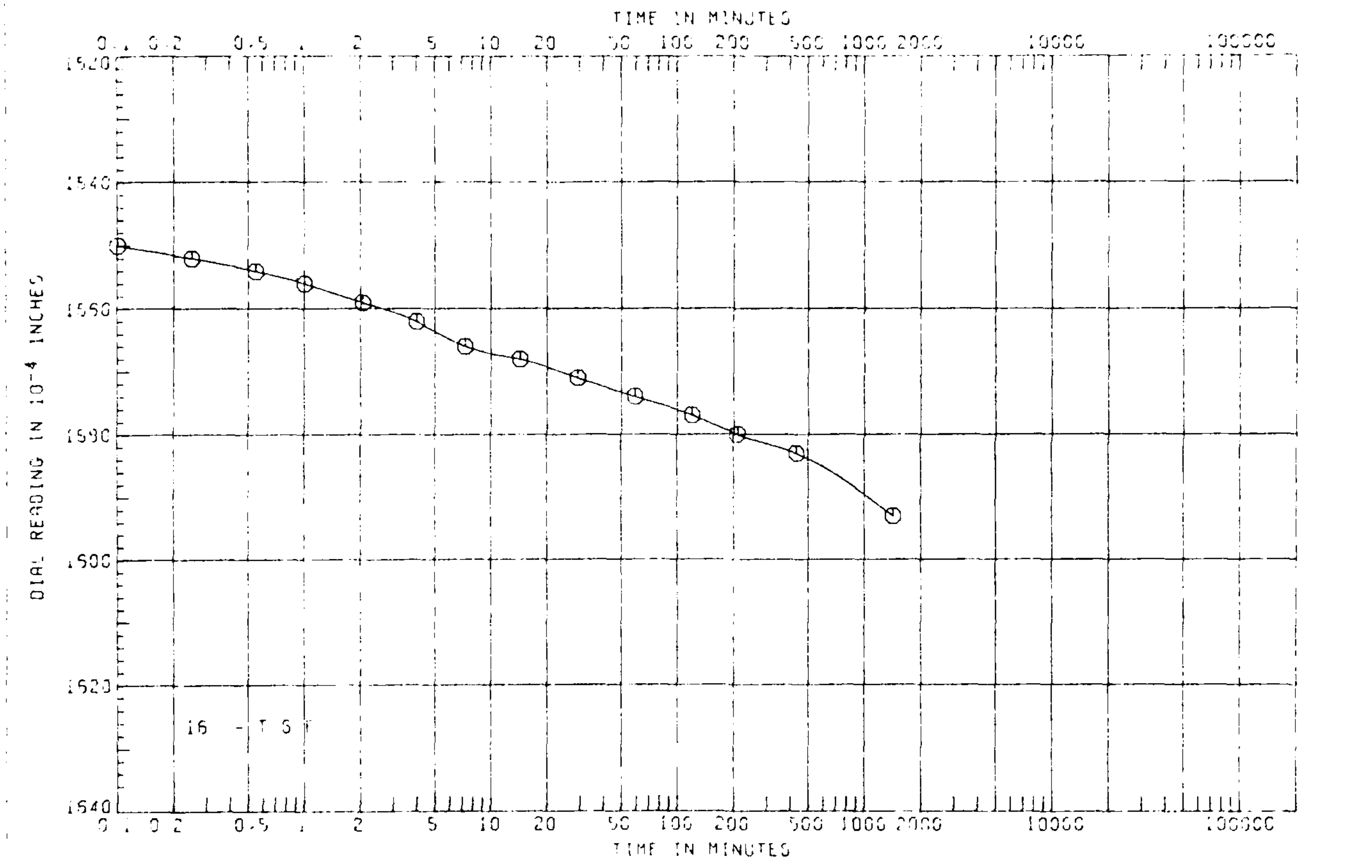
PROJECT DETROIT SEWER TUNNEL

BORING WES-3 SAMPLE NO. 3

DEPTH/ELEV 55. -55.5 DATE 06 JUN 80

SHEET 7 OF 13

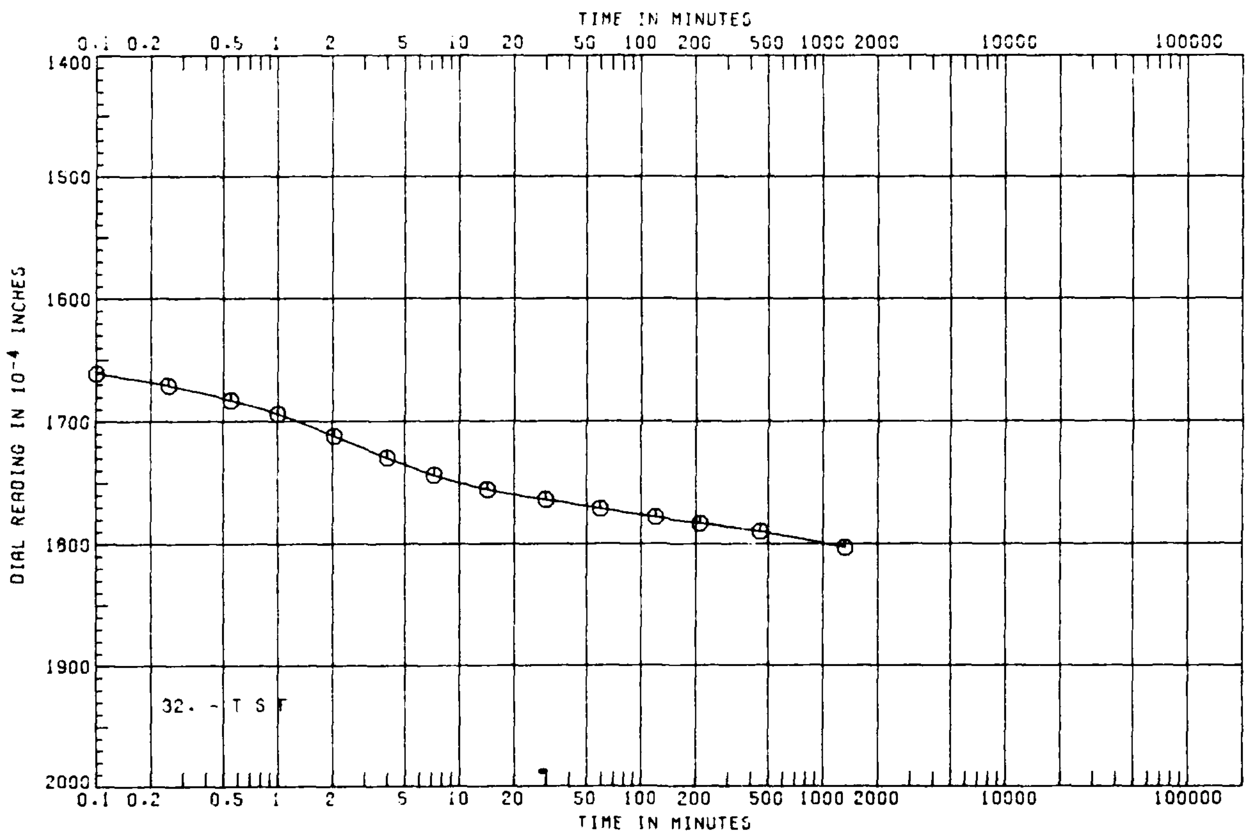
CONSOLIDATION TEST
TIME CURVES



PROJECT DETROIT SEWER TUNNEL	
BORING WES-3	SAMPLE NO 19
DEPTH/ELEV 55.1-55.5	DATE 05 JUN 85
SHEET 8 OF 13	

CONSOLIDATION TEST

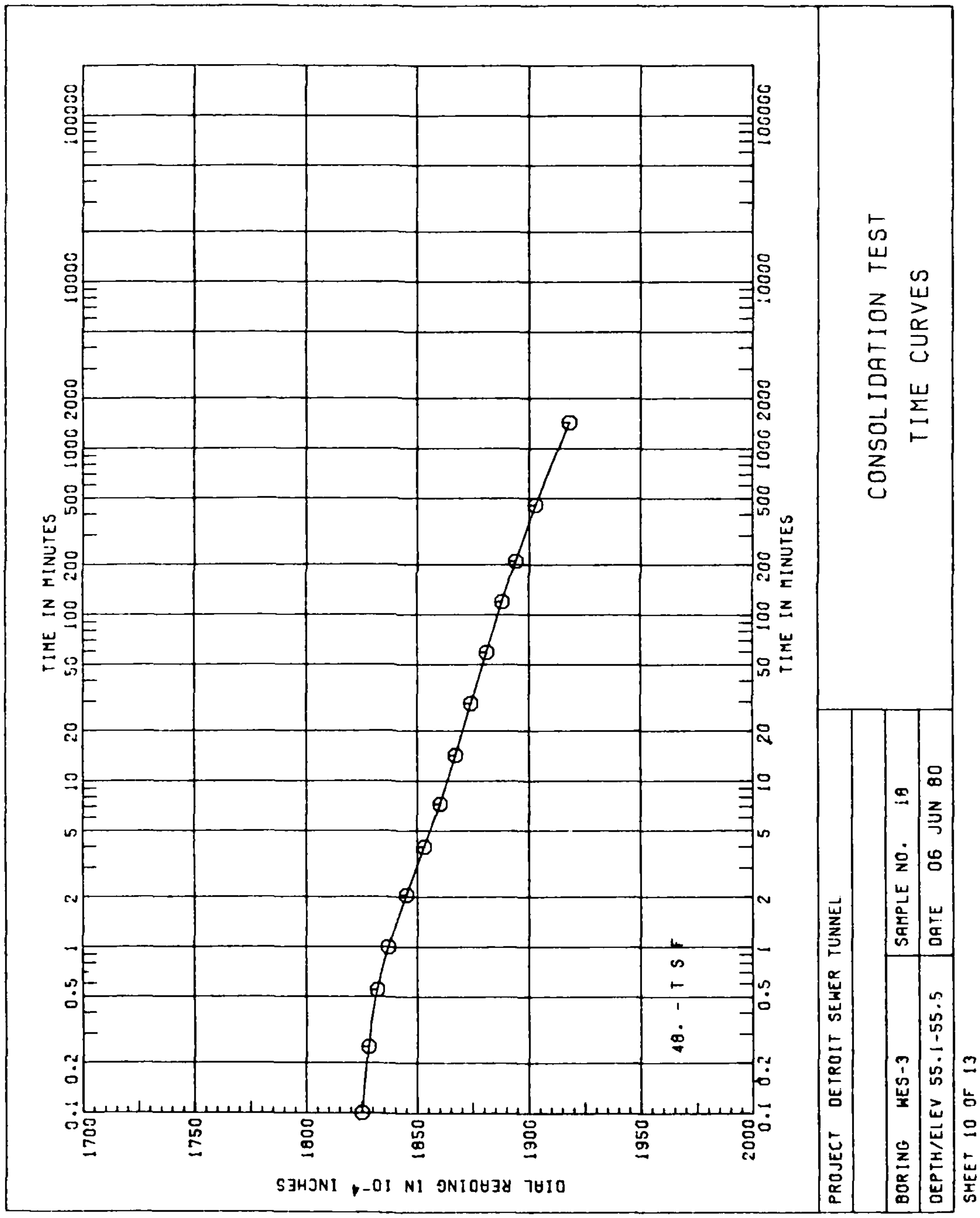
TIME CURVES

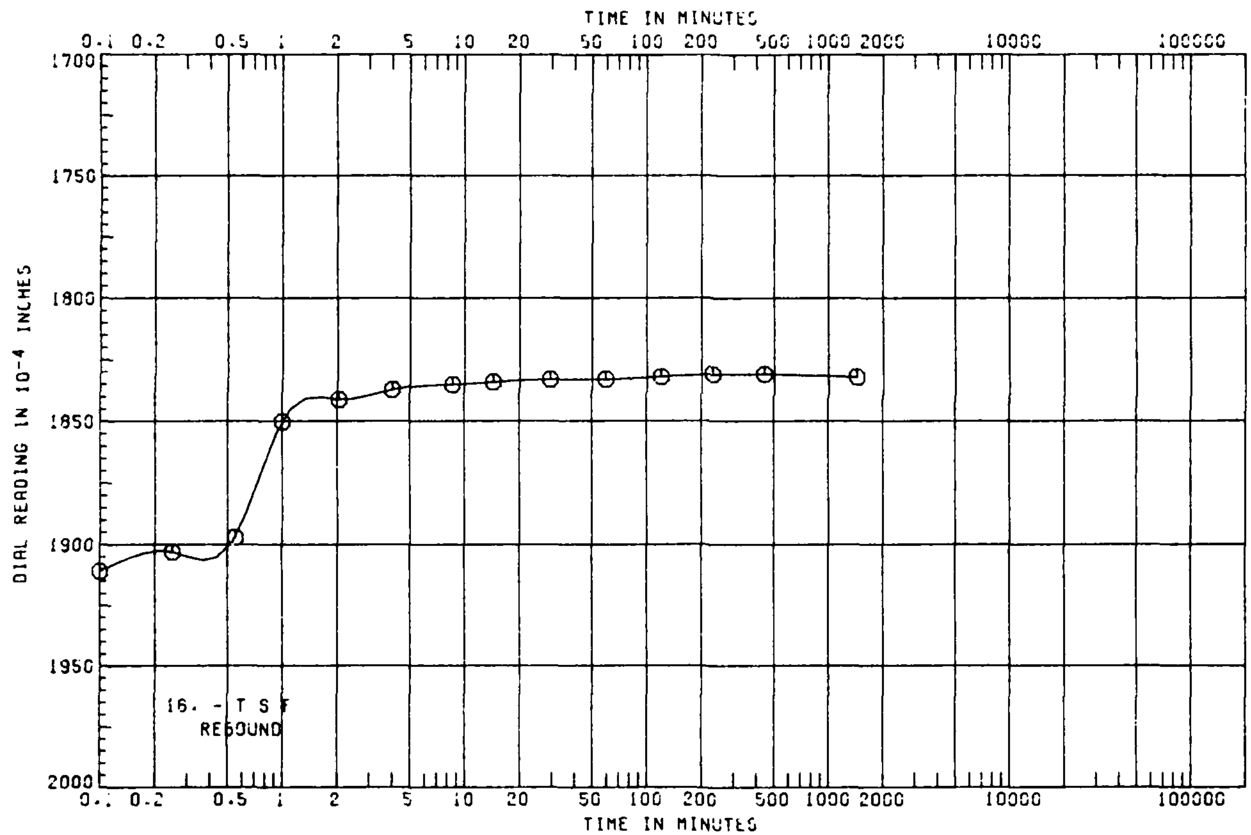


PROJECT DETROIT SEWER TUNNEL	
BORING WES-3	SAMPLE NO 18
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 80

CONSOLIDATION TEST
TIME CURVES

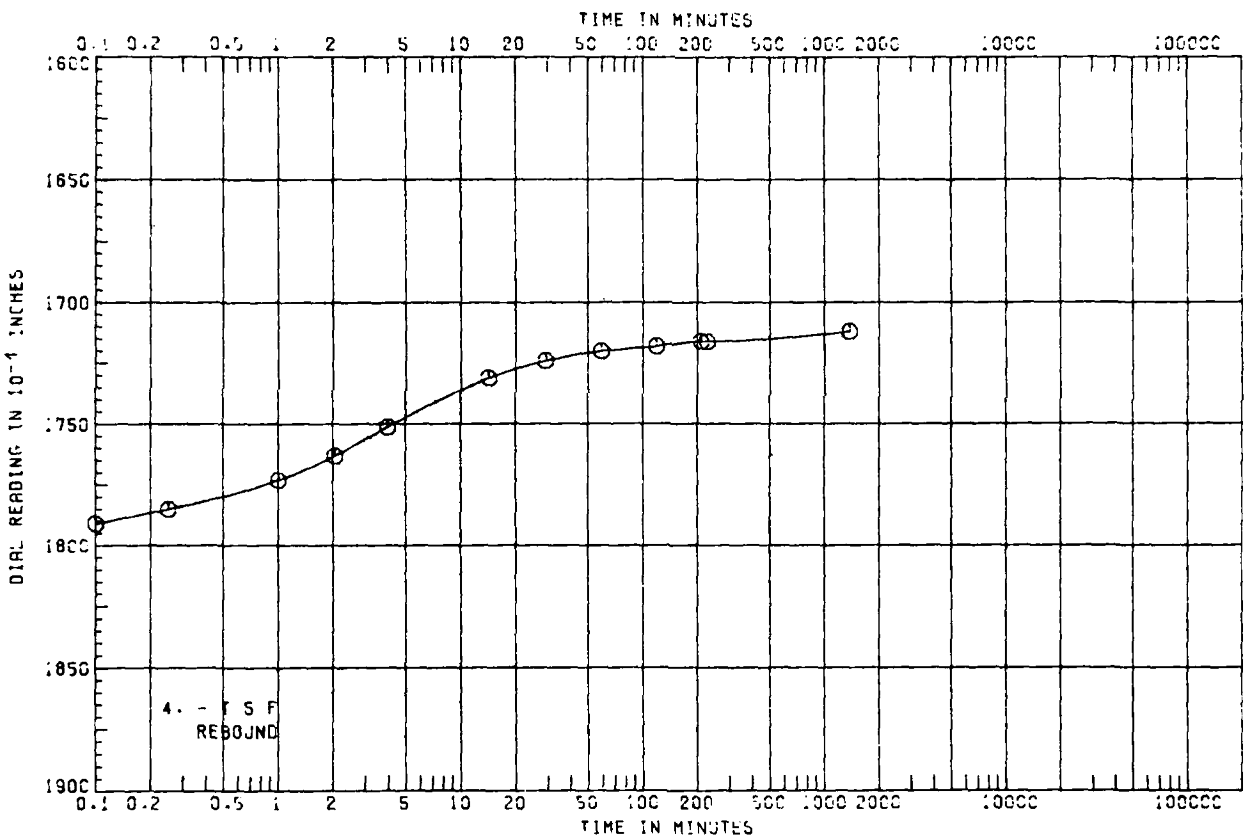
SHEET 9 OF 13


CONSOLIDATION TEST
TIME CURVES
TIME IN MINUTES
0.1 0.2 0.5 1 2 5 10 20 50 100 200 500 1000 2000 10000 100000
DIAL READING IN 10^{-4} INCHES
1700 1750 1800 1850 1900 1950 2000
48. - T S F
PROJECT DETROIT SEWER TUNNEL
BORING WES-3
SAMPLE NO. 18
DEPTH/ELEV 55.1-55.5
DATE 06 JUN 80
SHEET 10 OF 13



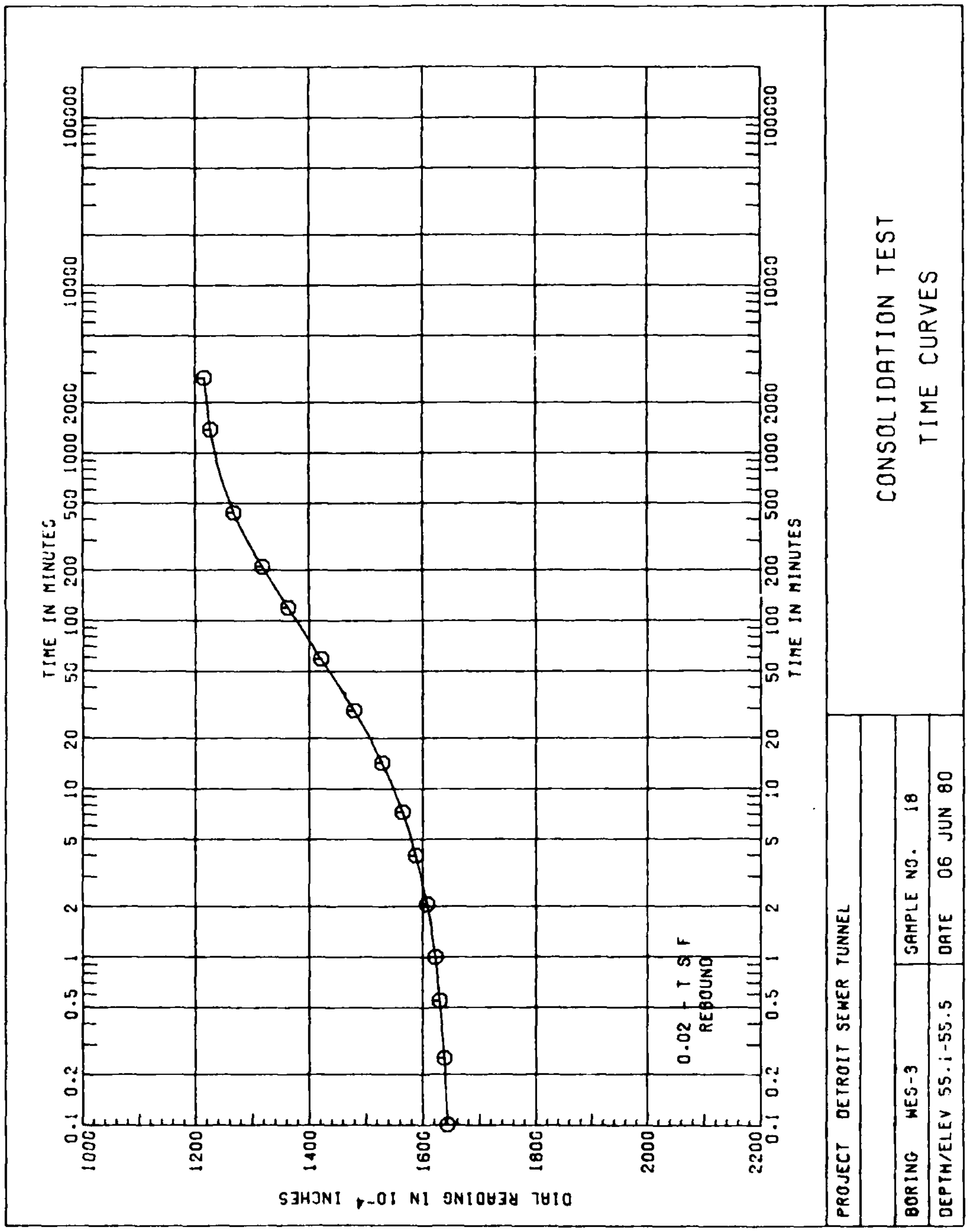
PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-3	SAMPLE NO. 18	
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 80	

SHEET 11 OF 13

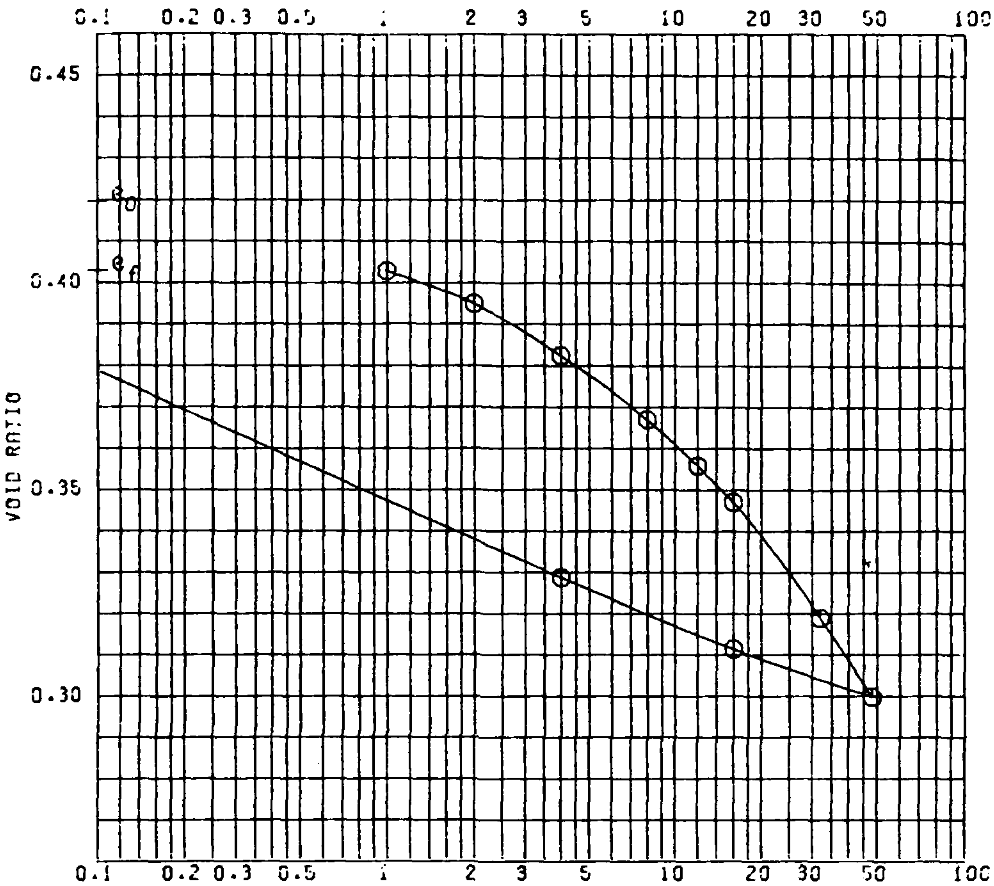


PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-3	SAMPLE NO. 18	
DEPTH/ELEV 55.1-55.5	DATE 06 JUN 80	

SHEET 12 OF 13



CONSOLIDATION TEST
TIME CURVES
TIME IN MINUTES
DIAL READING IN 10-4 INCHES
0.02 - T S F
REBOUND
PROJECT DETROIT SEWER TUNNEL
BORING WES-3
SAMPLE NO. 18
DEPTH/ELEV 55.1-55.5
DATE 06 JUN 80
SHEET 13 OF 13



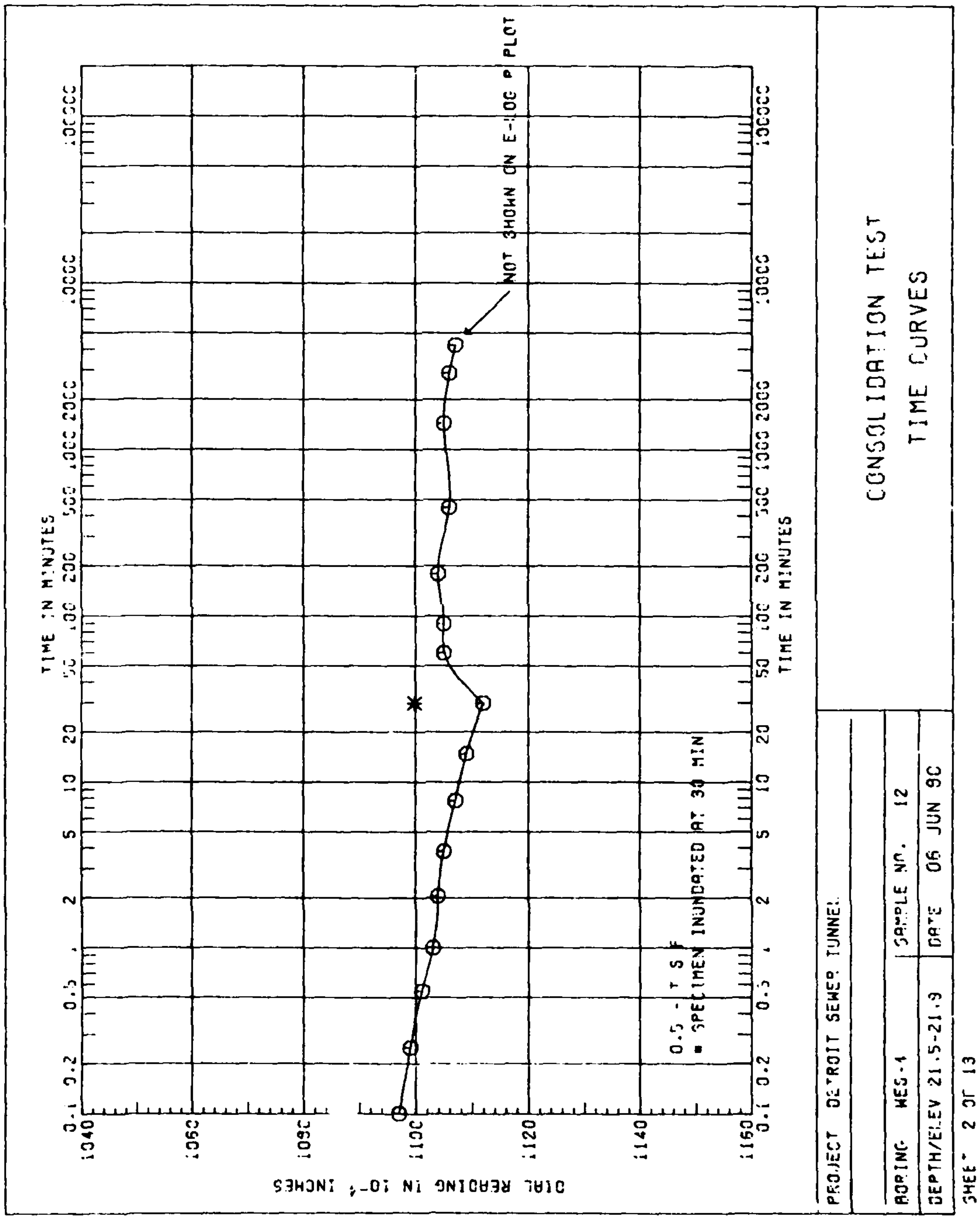
			BEFORE TEST	AFTER TEST
OVERBURDEN PRESSURE, TSF		WATER CONTENT, %	14.0	14.5
PRECONSOL. PRESSURE, TSF		DRY DENSITY, PCF	121.4	122.9
COMPRESSION INDEX		SATURATION, %	92.3	99.6
TYPE SPECIMEN	UNDISTURBED	VOID RATIO	0.419	0.403
DIA. IN 4.44	HT. IN 1.127	BACK PRESSURE, TSF		

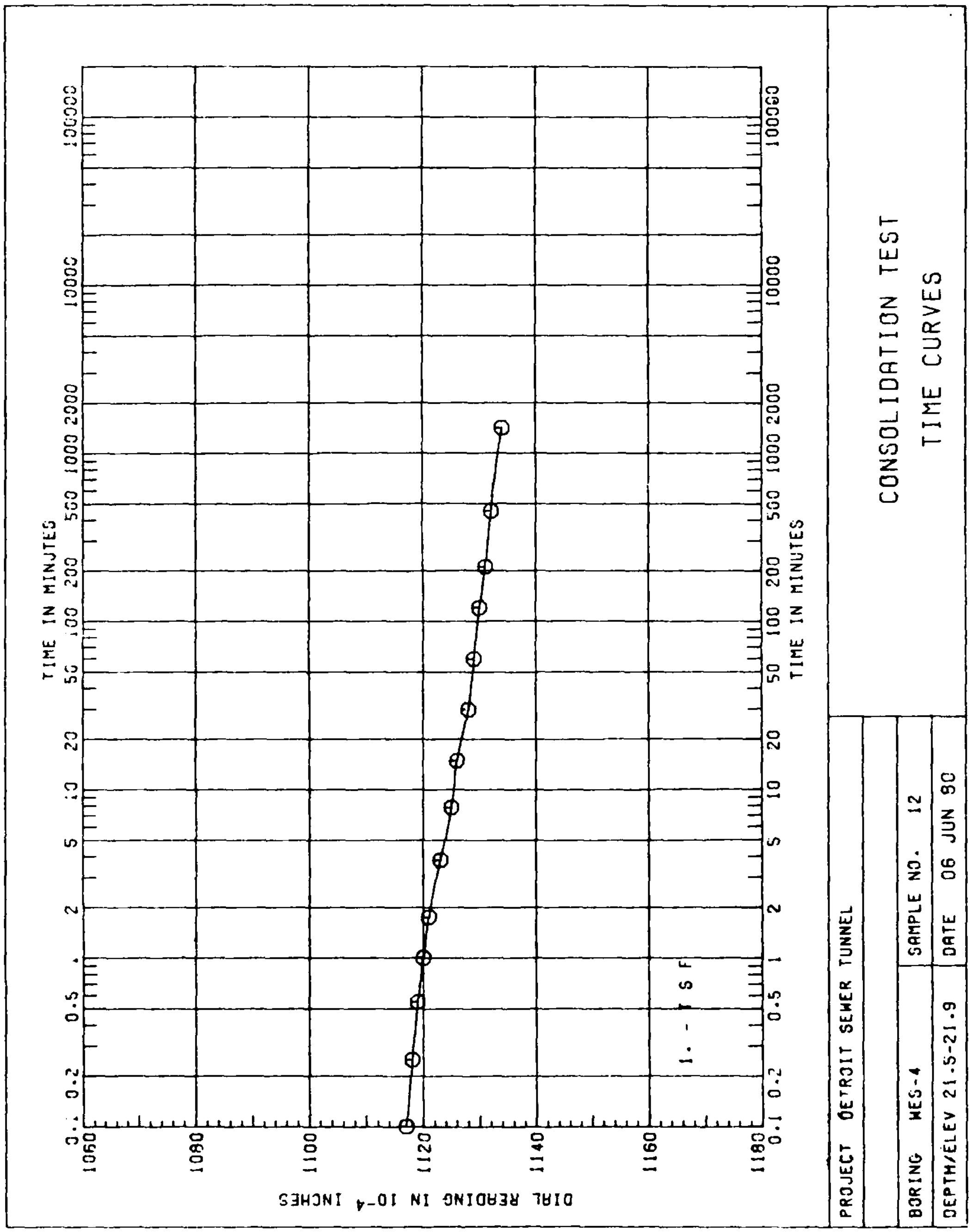
CLASSIFICATION CLAY (CL), BROWNISH GRAY; GRAVEL TO 1/4"

LL 29	PL 16	PI 13	PROJECT DETROIT SEWER TUNNEL
GS 2.76 COM	D_{10}		
REMARKS		BORING NO. WES-4	SAMPLE NO. 12
		DEPTH/ELEV 21.5-21.9	DATE 06 JUN 90

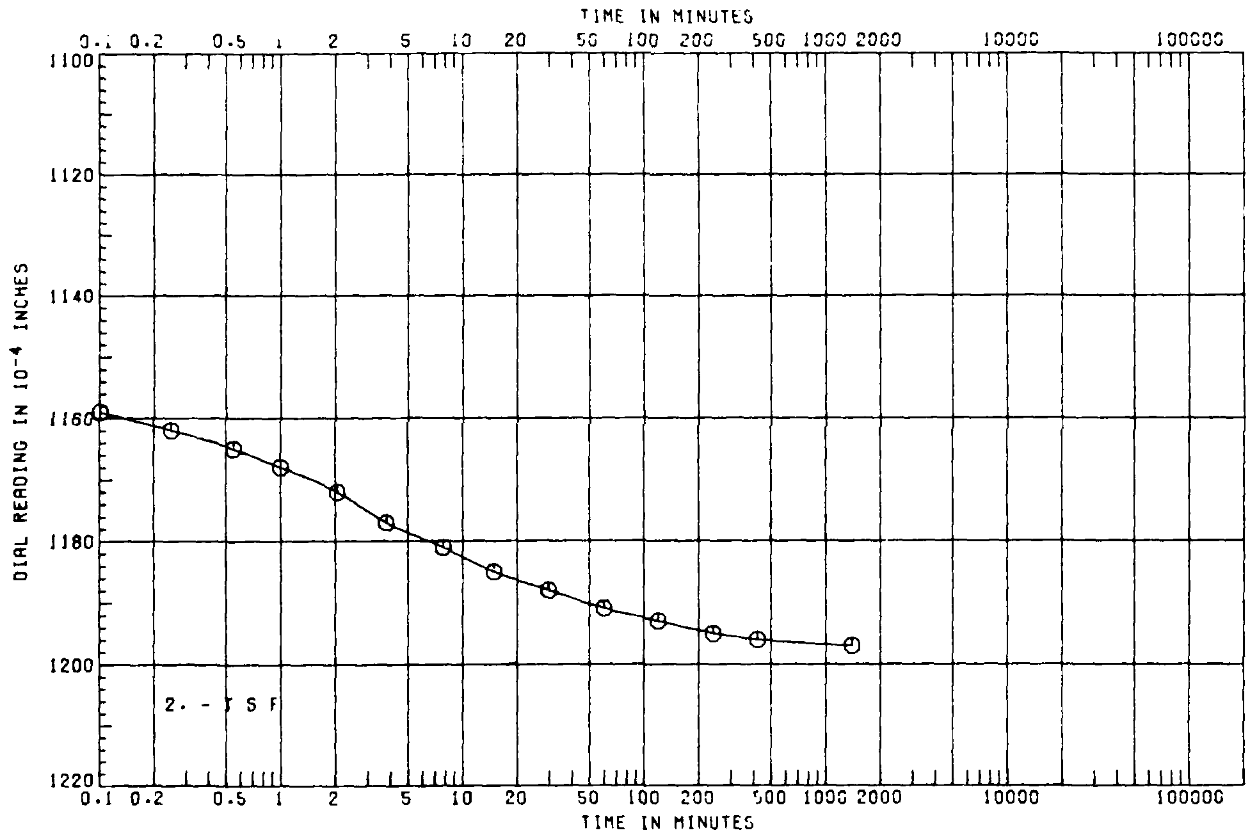
CONSOLIDATION TEST REPORT

SHEET 1 OF 13


CONSOLIDATION TEST
TIME CURVES
TIME IN MINUTES
DIAL READING IN 10⁻⁴ INCHES
NOT SHOWN ON E-LOG P PLOT
0.5 - T S F
* SPECIMEN INUNDATED AT 30 MIN
PROJECT DETROIT SEWER TUNNEL
BORING WES-4
SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9
DATE 06 JUN 90
SHEET 2 OF 13



CONSOLIDATION TEST
TIME CURVES
TIME IN MINUTES
DIAL READING IN 10-4 INCHES
1. - T S F
PROJECT DETROIT SEWER TUNNEL
BORING WES-4
SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9
DATE 06 JUN 90
SHEET 3 OF 13

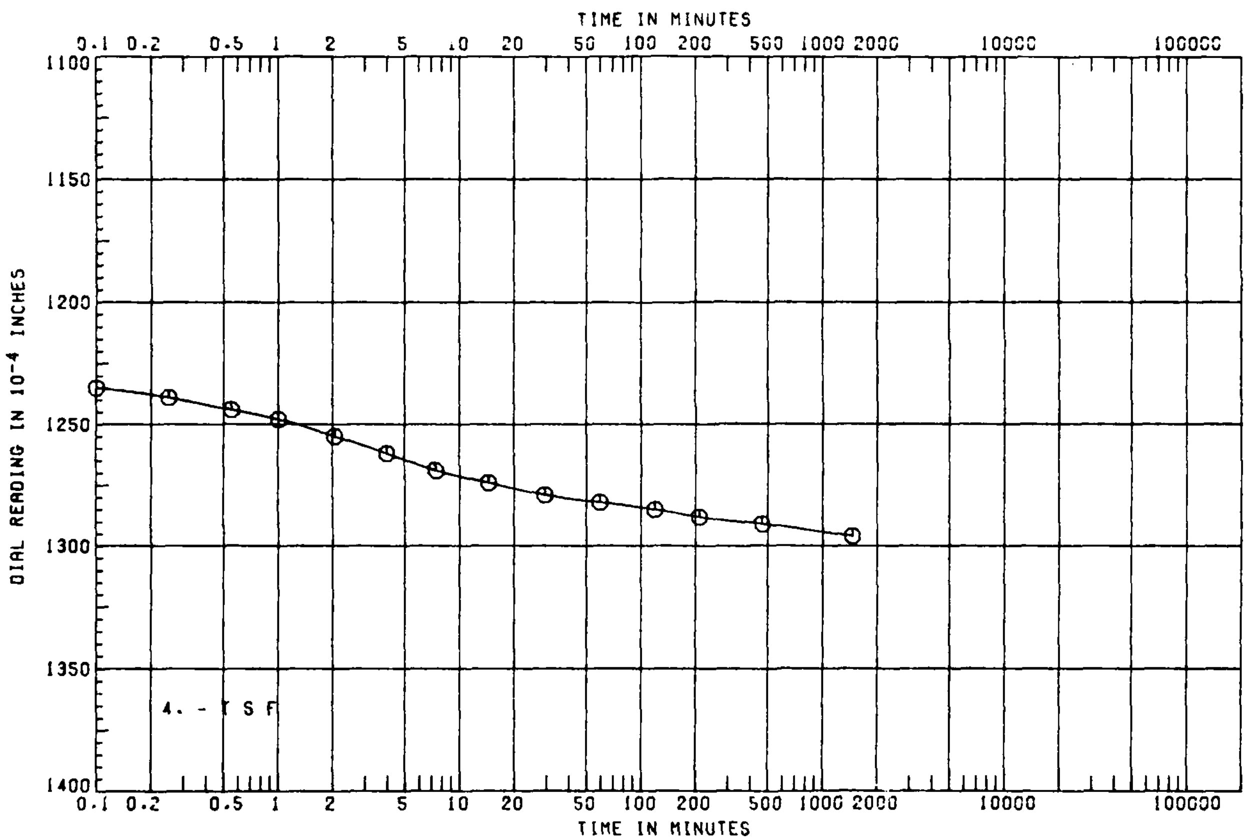


PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 90

CONSOLIDATION TEST

TIME CURVES

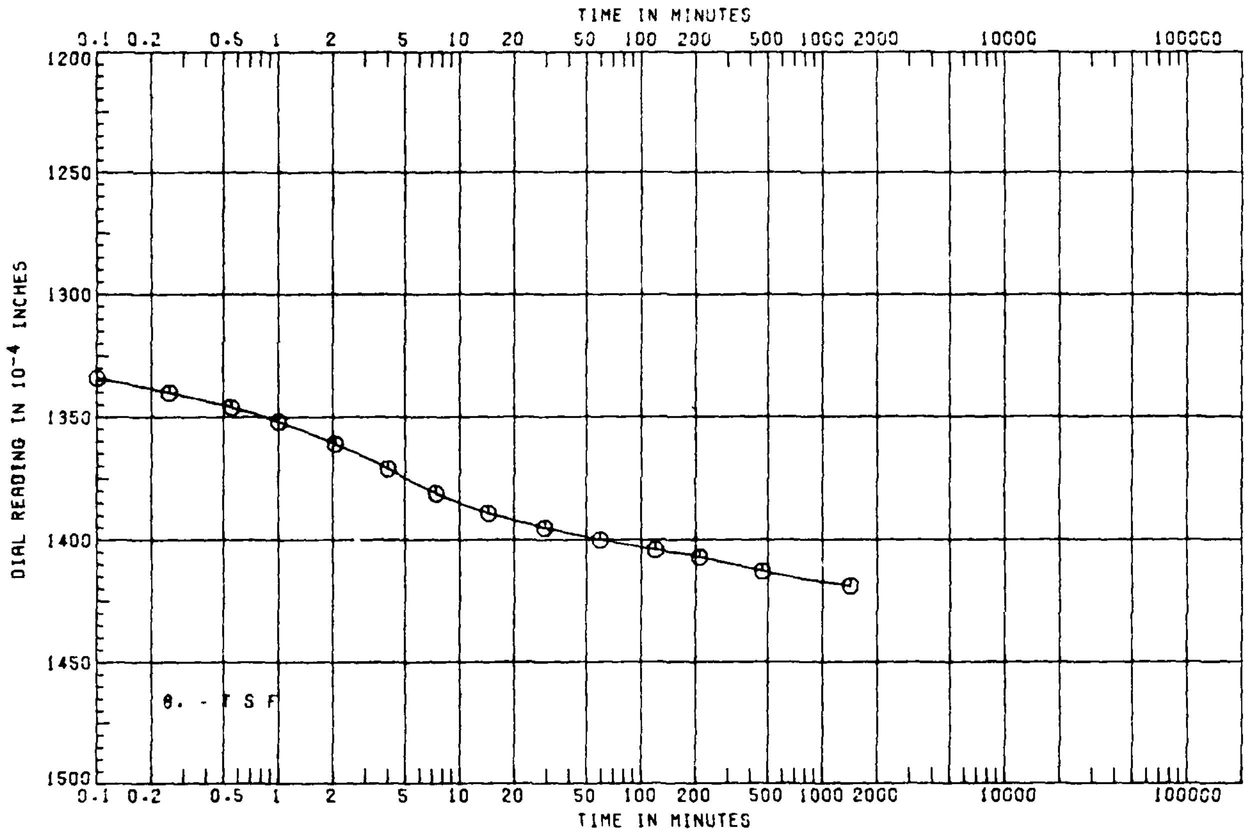
SHEET 4 OF 13



PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80

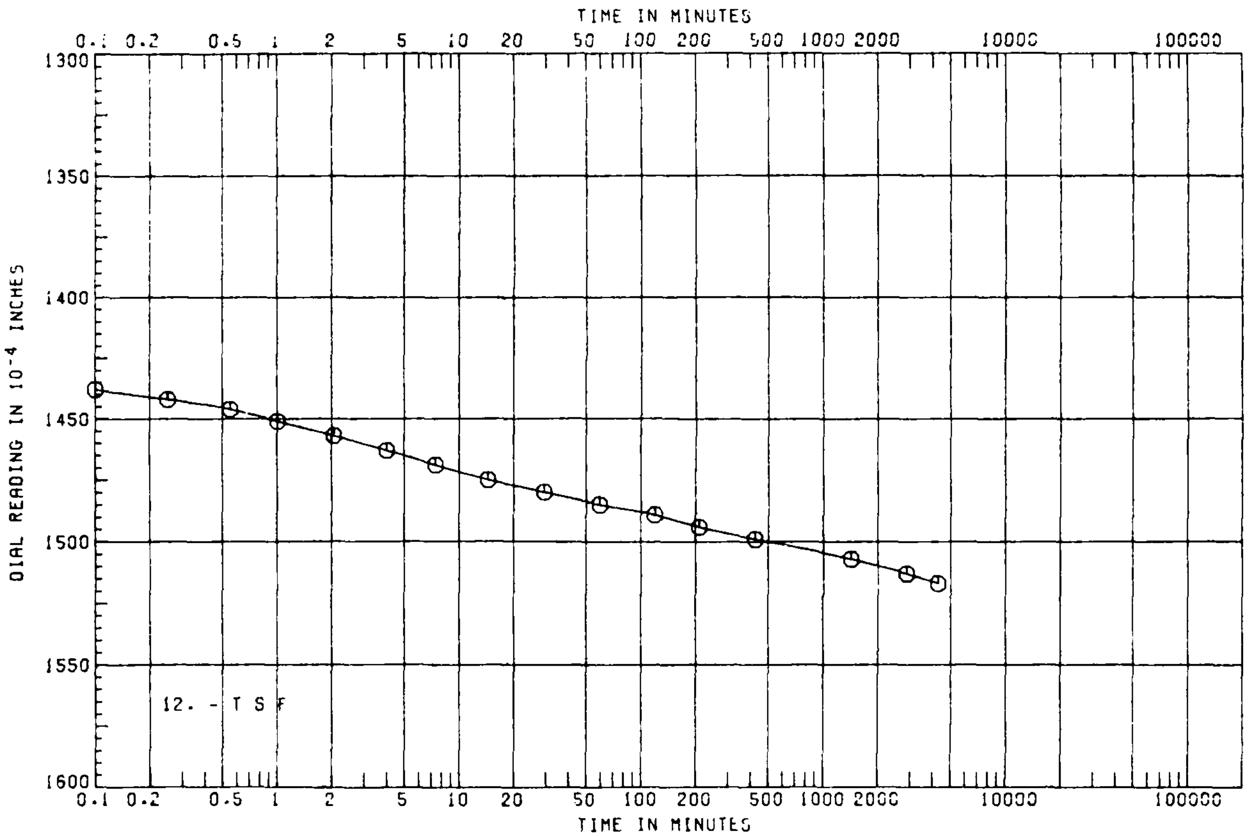
CONSOLIDATION TEST
TIME CURVES

SHEET 5 OF 13



PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-4	SAMPLE NO. 12	
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80	

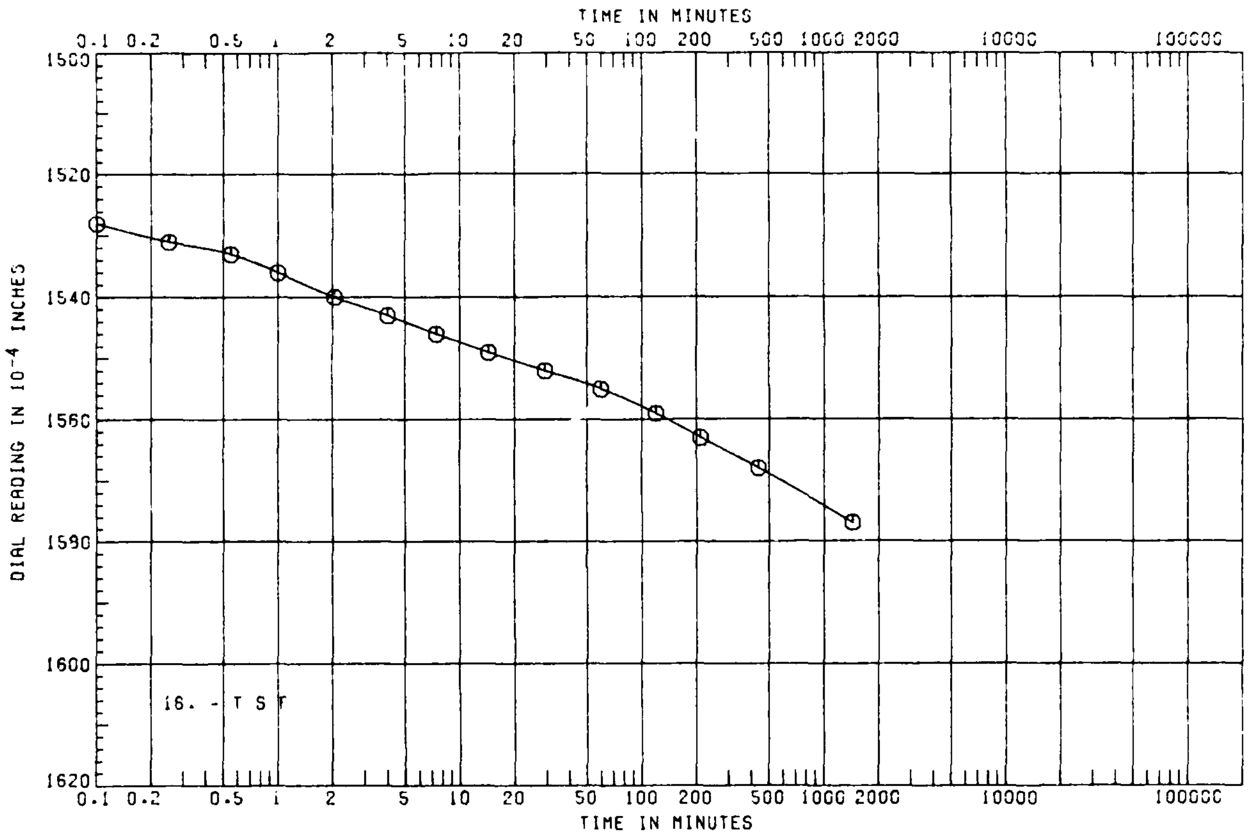
SHEET 6 OF 13



PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80

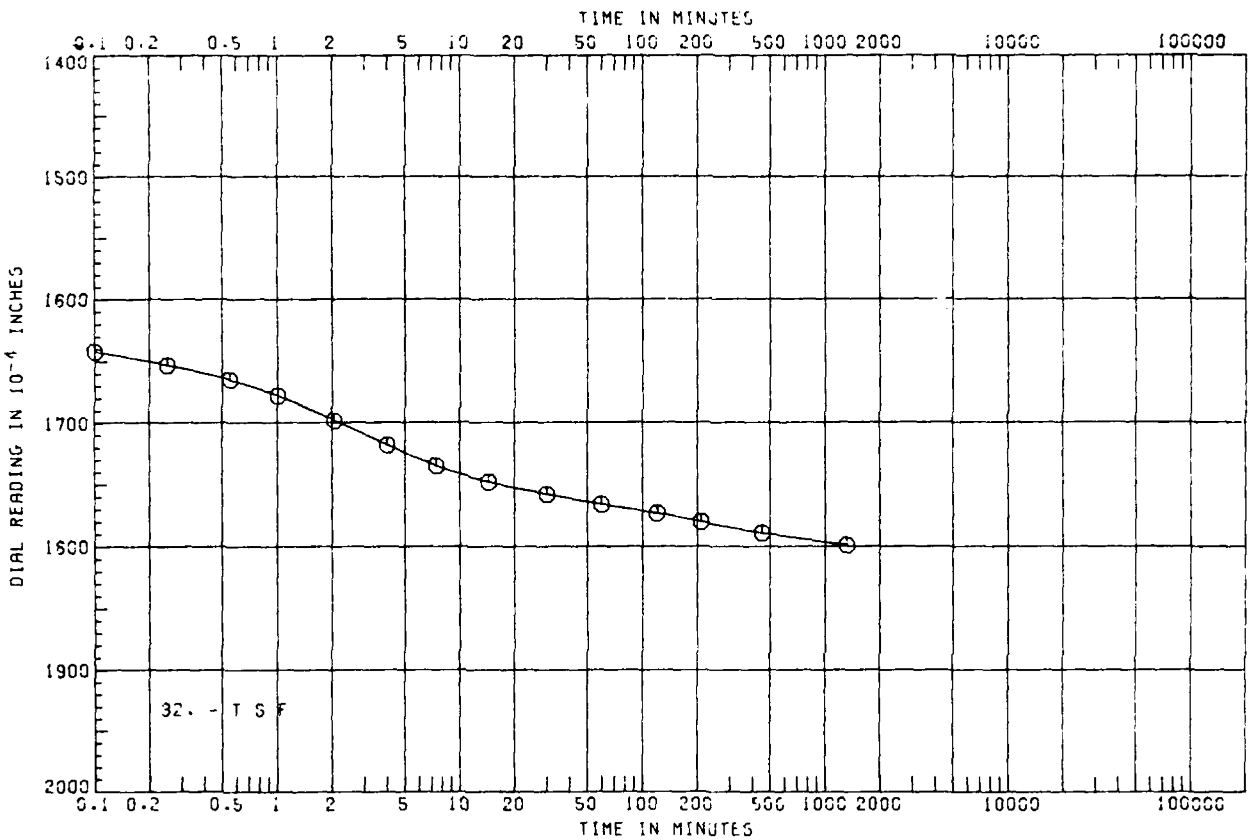
CONSOLIDATION TEST
TIME CURVES

SHEET 7 OF 13



PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-4	SAMPLE NO. 12	
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80	

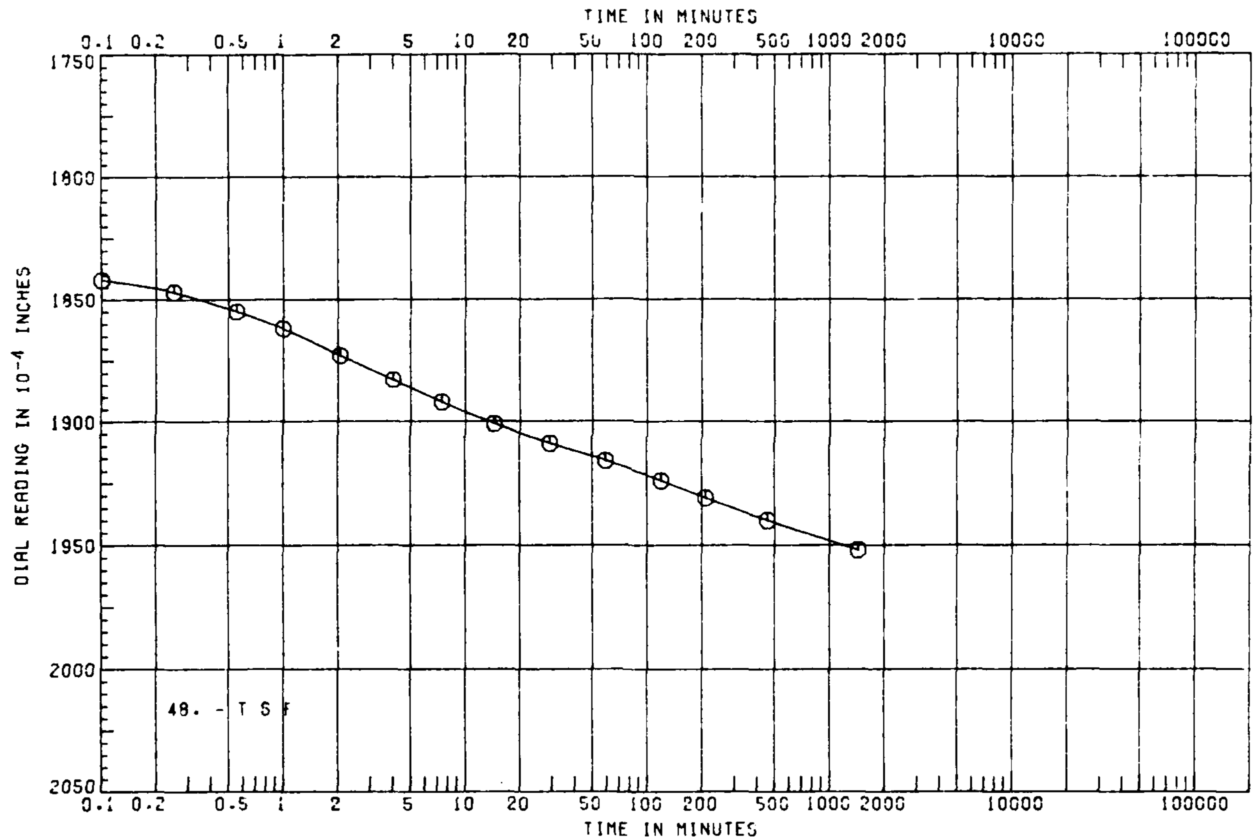
SHEET 9 OF 13



PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80

CONSOLIDATION TEST
TIME CURVES

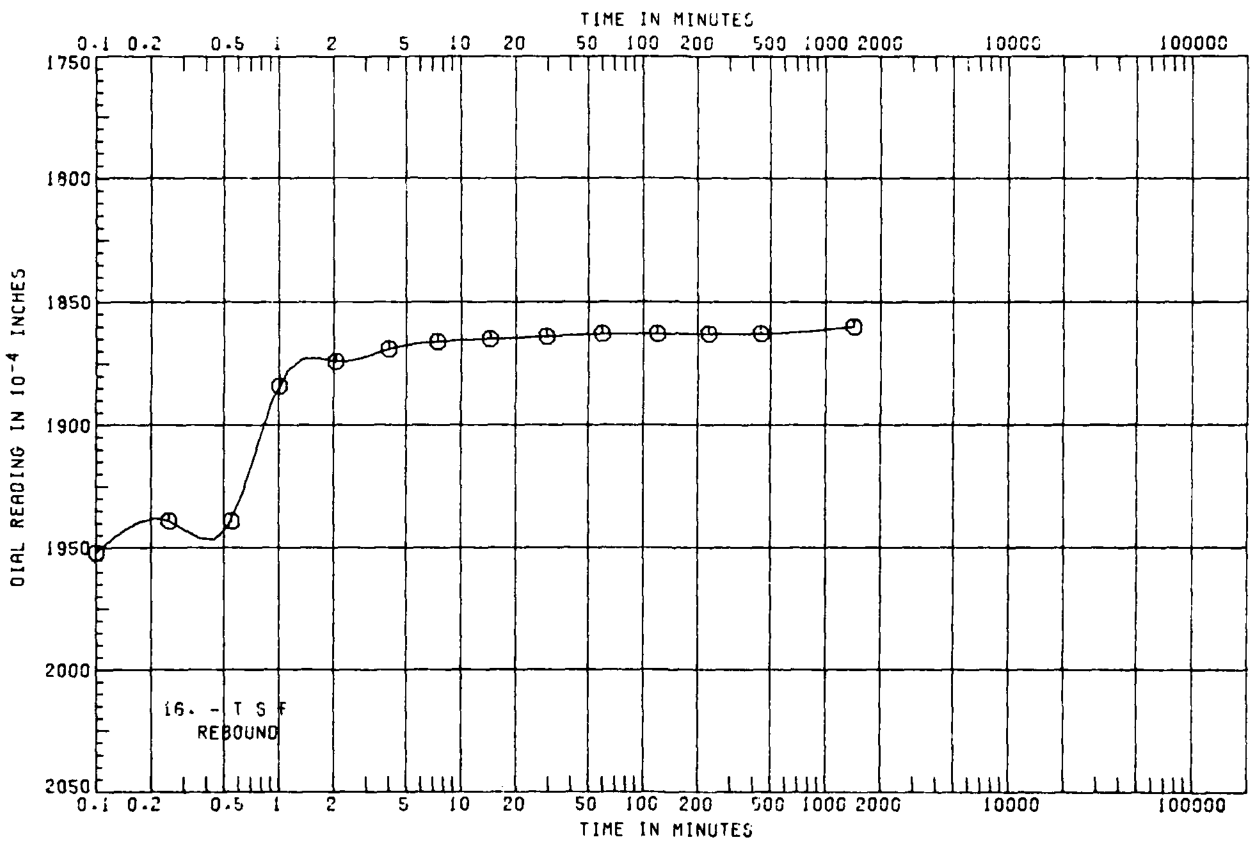
SHEET 9 OF 13



PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80

CONSOLIDATION TEST
TIME CURVES

SHEET 10 OF 13

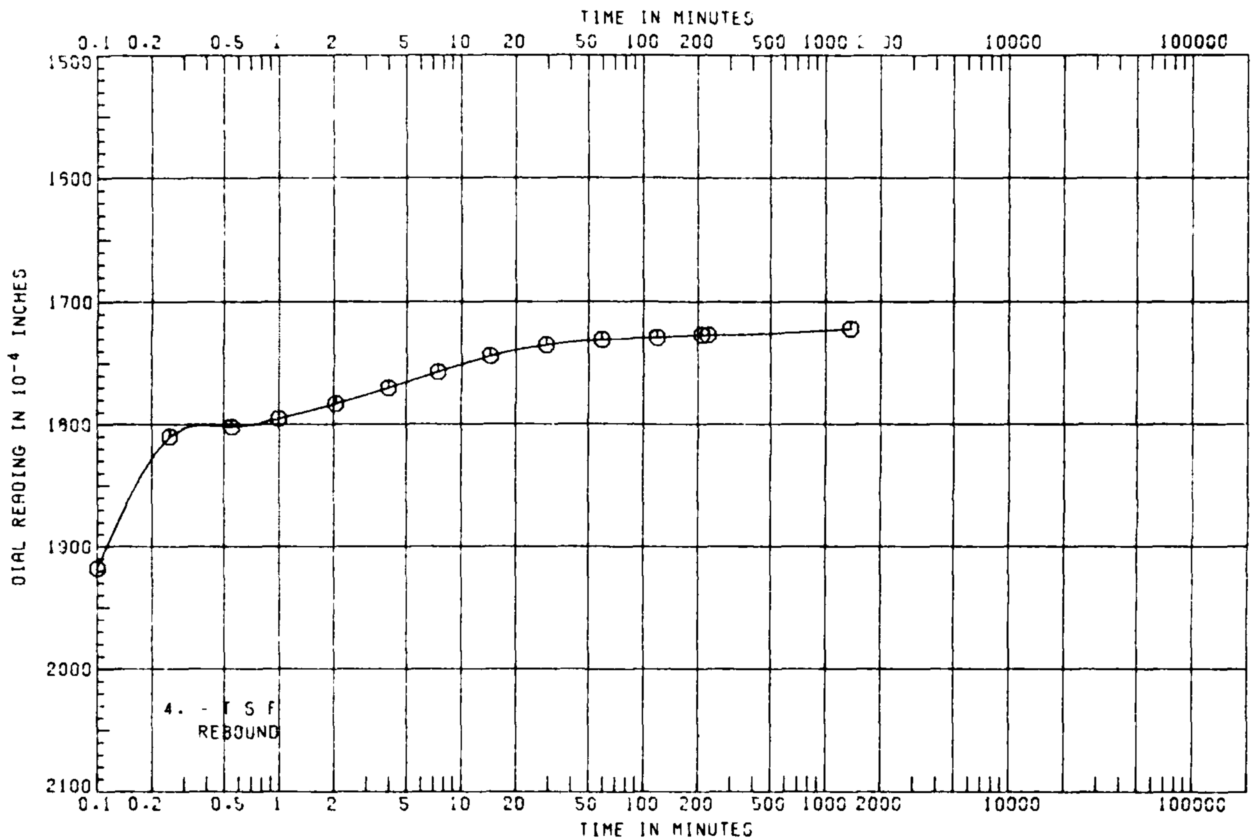


PROJECT DETROIT SEWER TUNNEL	
BORING WES-4	SAMPLE NO. 12
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 90

CONSOLIDATION TEST

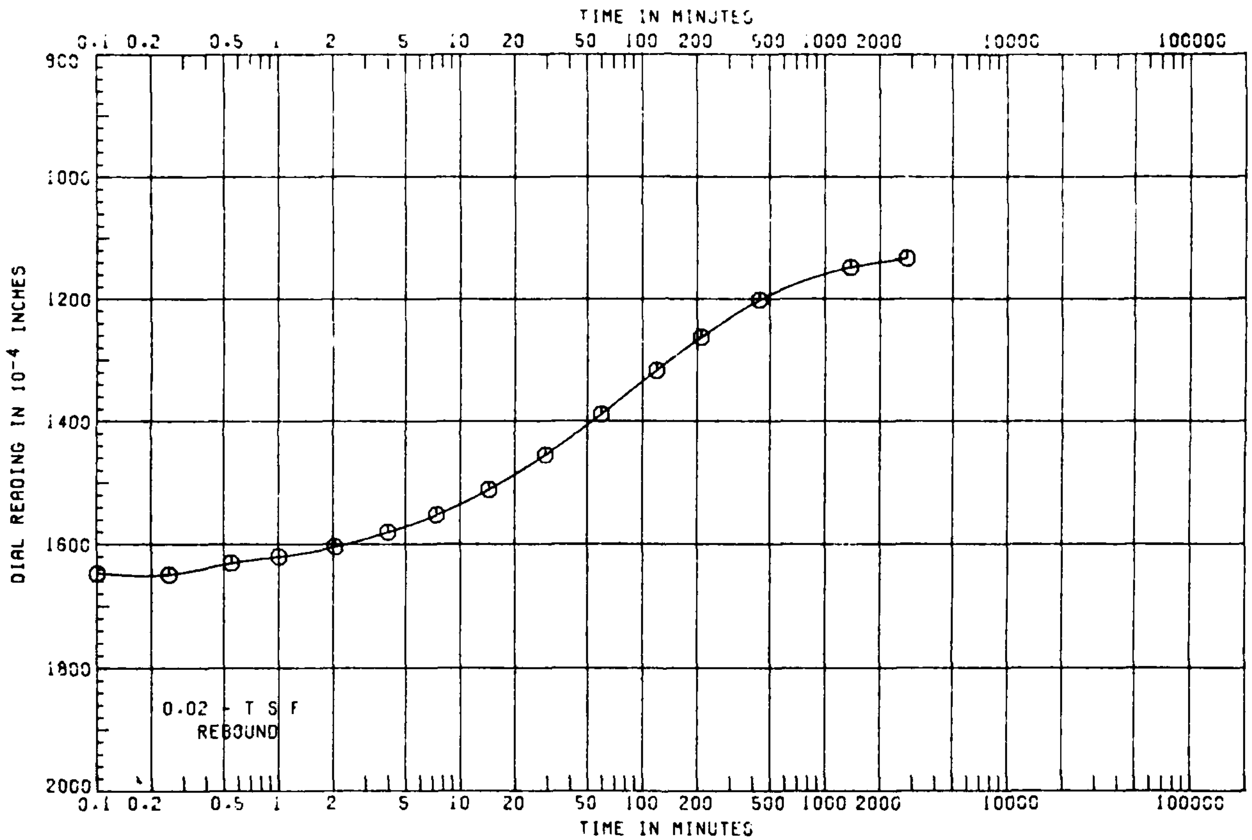
TIME CURVES

SHEET 11 OF 13



PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-4	SAMPLE NO. 12	
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80	

SHEET 12 OF 13

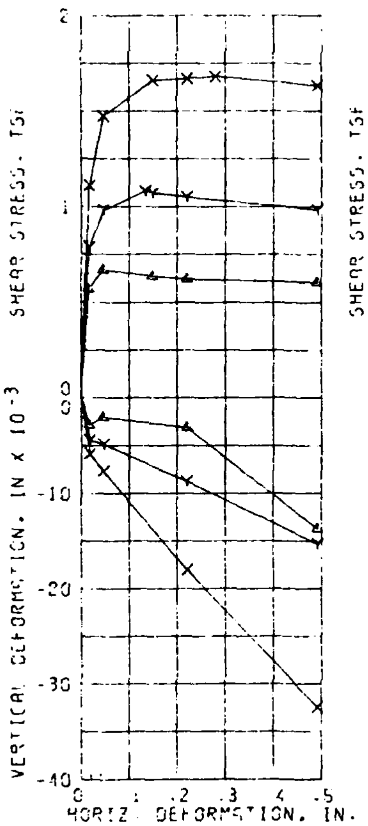


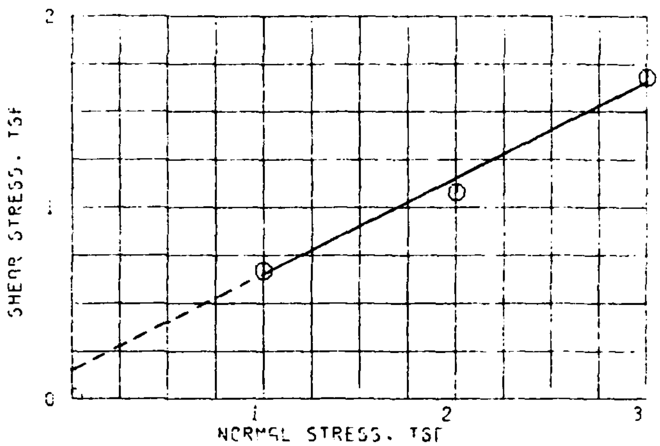
PROJECT DETROIT SEWER TUNNEL		CONSOLIDATION TEST TIME CURVES
BORING WES-4	SAMPLE NO. 12	
DEPTH/ELEV 21.5-21.9	DATE 06 JUN 80	

SHEET 13 OF 13

APPENDIX D

DIRECT SHEAR TEST DATA





Φ = 26.5°

TAN Φ = 0.50

C = 0.15 TSF

		TEST NO.	1 △	2 ⅄	3 ×
INITIAL		WATER CONTENT, %	14.6	14.0	13.9
		VOID RATIO	0.381	0.391	0.377
		SATURATION, %	100 +	99.0	100 +
		DRY DENSITY, PCF	124.7	123.9	125.1
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL, MIN			< 1	< 1	< 1
FINAL		WATER CONTENT, %	14.2	14.9	13.9
		VOID RATIO			
		SATURATION, %			
NORMAL STRESS, TSF			1.0	2.0	3.0
MAXIMUM SHEAR STRESS, TSF			0.67	1.09	1.69
TIME TO FAILURE, MIN			251	757	1570
RATE OF STRAIN, IN/MIN			.00019	.00019	.00019
ULTIMATE SHEAR STRESS, TSF					

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	0.594 IN. THICK

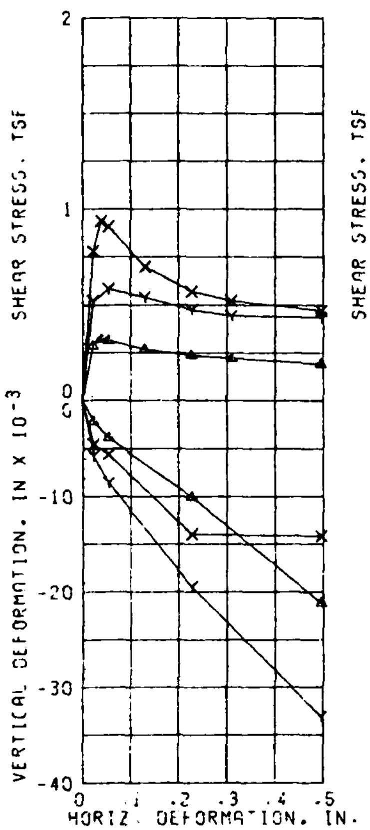
CLASSIFICATION SANDY CLAY (CL), GRAY, GRAVEL TO 1/2"

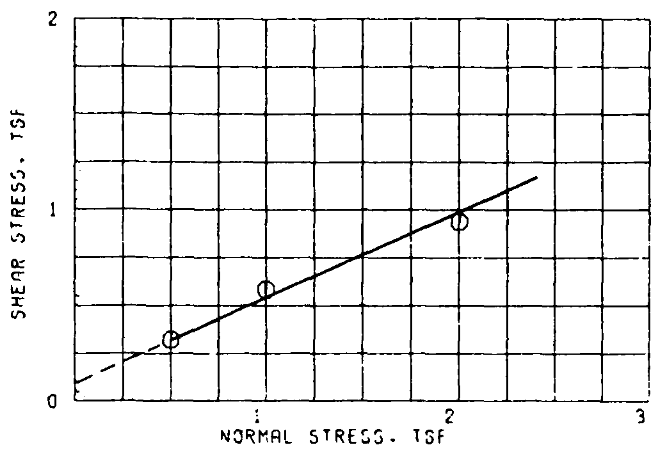
LL 22	PL 11	PI 11		GS 2.76

REMARKS.

PROJECT DETROIT SEWER TUNNEL	
BORING NO. WES-3	SAMPLE 9
DEPTH/ELEV 41.1/42.1	DATE 06 JUN 90

DIRECT SHEAR TEST REPORT





ϕ = 24.0°

TAN ϕ = 0.44

C = 0.10 TSF

	TEST NO.	1 △	2 Y	3 X
INITIAL	WATER CONTENT, %	29.3	32.1	30.3
	VOID RATIO	0.797	0.779	0.790
	SATURATION, %	100 +	100 +	100 +
	DRY DENSITY, PCF	95.7	96.1	95.5
VOID RATIO AFTER CONSOL				
FIFTY PERCENT CONSOL, MIN		2	< 1	< 1
FINAL	WATER CONTENT, %	32.2	27.0	29.9
	VOID RATIO			
	SATURATION, %			
NORMAL STRESS, TSF		0.5	1.0	2.0
MAXIMUM SHEAR STRESS, TSF		0.32	0.59	0.94
TIME TO FAILURE, MIN		214	302	214
RATE OF STRAIN, IN/MIN		.00019	.00019	.00019
ULTIMATE SHEAR STRESS, TSF				

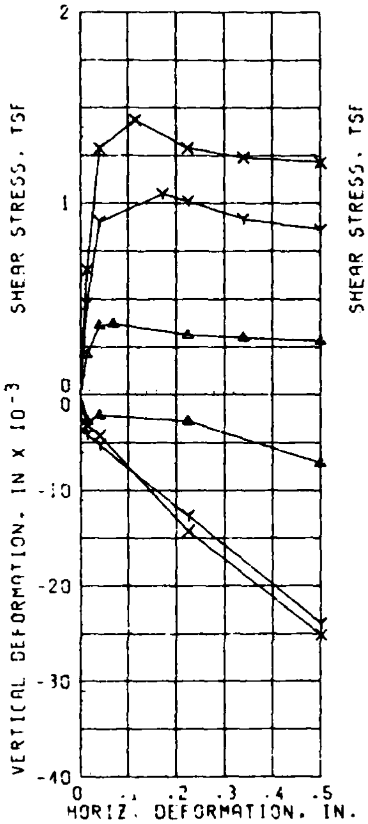
TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	0.594 IN. THICK

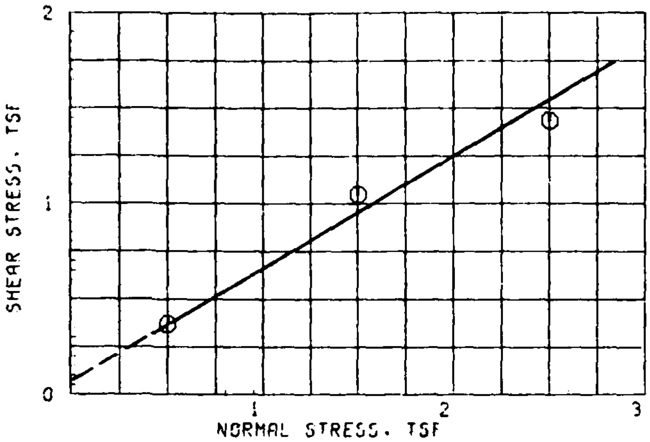
CLASSIFICATION CLAY (CL), GRAY; TRACE OF SILT

LL 32	PL 18	PI 14		GS 2.74

REMARKS.	PROJECT DETROIT SEWER TUNNEL
	BORING NO. WES-4 — SAMPLE 9
	DEPTH/ELEV 10.2/11.3 — DATE 24 JUN 80

DIRECT SHEAR TEST REPORT





Φ = 30.5°

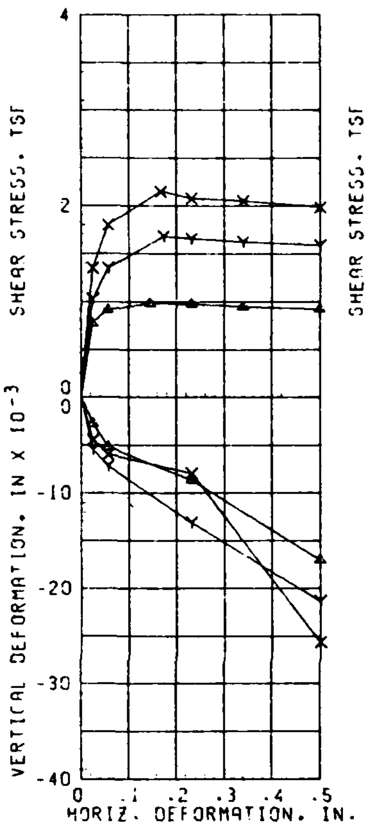
TAN Φ = 0.59

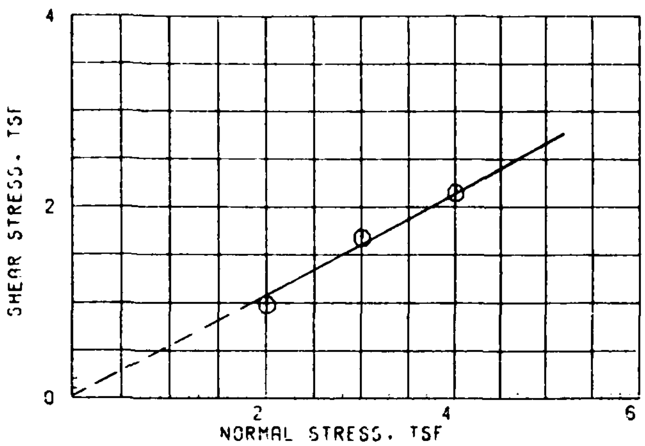
C = 0.08 TSF

		TEST NO.	1 △	2 Y	3 X	
INITIAL		WATER CONTENT, %	15.5	14.4	14.3	
		VOID RATIO	0.396	0.377	0.375	
		SATURATION, %	100 +	100 +	100 +	
		DRY DENSITY, PCF	122.9	124.6	124.9	
VOID RATIO AFTER CONSOL						
FIFTY PERCENT CONSOL, MIN			< 1	4	3	
FINAL		WATER CONTENT, %	17.4	15.2	15.9	
		VOID RATIO				
		SATURATION, %				
NORMAL STRESS, TSF			0.5	1.5	2.5	
MAXIMUM SHEAR STRESS, TSF			0.37	1.05	1.44	
TIME TO FAILURE, MIN			396	950	536	
RATE OF STRAIN, IN/MIN			.00019	.00019	.00019	
ULTIMATE SHEAR STRESS, TSF						

TYPE SPECIMEN UNDISTURBED			3.00 IN. SQUARE	0.594 IN. THICK
CLASSIFICATION CLAY (CL), GRAY; MED. SAND TO FINE GRAVEL				
LL 31	PL 16	PI 15		GS 2.75

REMARKS:	PROJECT DETROIT SEWER TUNNEL	
	BORING NO. WES-4	SAMPLE 13
	DEPTH/ELEV 25.3/25.2	DATE 27 JUN 90
	DIRECT SHEAR TEST REPORT	





Φ = 28.5°

TAN Φ = 0.54

C = 0

	TEST NO.	1 △	2 Y	3 X	
INITAL	WATER CONTENT. %	15.5	16.2	16.9	
INITAL	VOID RATIO	0.349	0 344	0 341	
INITAL	SATURATION. %	100 +	100 +	100 +	
INITAL	DRY DENSITY. PCF	126.9	127.3	127.5	
VOID RATIO AFTER CONSOL					
FIFTY PERCENT CONSOL. MIN		2	2	2	
FINAL	WATER CONTENT. %	15.8	13.7	12.9	
FINAL	VOID RATIO				
FINAL	SATURATION. %				
NORMAL STRESS. TSF		2.0	3.0	4.0	
MAXIMUM SHEAR STRESS. TSF		0 99	1.69	2.15	
TIME TO FAILURE. MIN		901	961	928	
RATE OF STRAIN. IN/MIN		.00019	.00019	.00019	
ULTIMATE SHEAR STRESS. TSF					

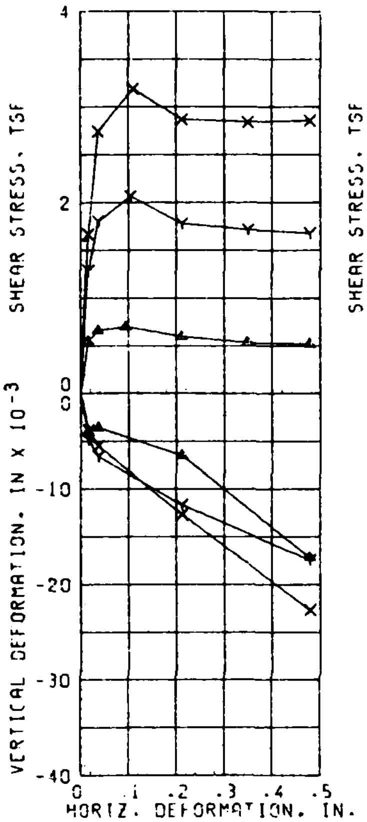
TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	0.594 IN. THICK

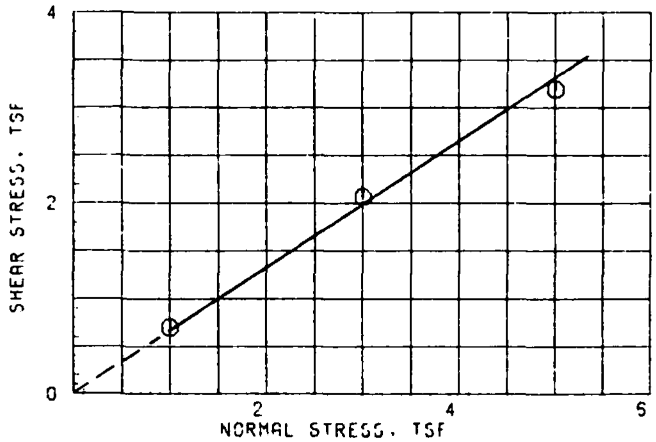
CLASSIFICATION SANDY CLAY (CL). WITH GRAVEL

LL 29	PL 12	PI 16		GS 2.74

REMARKS.	PROJECT DETROIT SEWER TUNNEL	
	BORING NO. WES-4	SAMPLE 24
	DEPTH/ELEV 50.0/51.1	DATE 02 JUL 90

DIRECT SHEAR TEST REPORT





ϕ = 33.5°

TAN ϕ = 0.66

C = 0

	TEST NO.	1 △	2 Y	3 X
INITIAL	WATER CONTENT, %	10.5	11.7	11.3
	VOID RATIO	0.294	0.319	0.311
	SATURATION, %	99.5	100 +	99.6
	DRY DENSITY, PCF	132.6	130.2	130.9
VOID RATIO AFTER CONSOL				
FIFTY PERCENT CONSOL, MIN		3	3	3
FINAL	WATER CONTENT, %	12.5	11.7	11.6
	VOID RATIO			
	SATURATION, %			
NORMAL STRESS, TSF		1.0	3.0	5.0
MAXIMUM SHEAR STRESS, TSF		0.71	2.07	3.20
TIME TO FAILURE, MIN		549	606	639
RATE OF STRAIN, IN/MIN		.00017	.00017	.00017
ULTIMATE SHEAR STRESS, TSF				

TYPE SPECIMEN UNDISTURBED	3.00 IN. SQUARE	0.584 IN. THICK

CLASSIFICATION SANDY CLAY (CL), GRAY, WITH GRAVEL;
CONCRETIONS TO 1/2"

LL 24	PL 12	PI 12		GS 2.75 (EST)

REMARKS:	PROJECT DETROIT SEWER TUNNEL	
	BORING NO. WES-4	SAMPLE 25
	DEPTH/ELEV 52.0/53.0	DATE 07 JUN 80

DIRECT SHEAR TEST REPORT

APPENDIX E

$K_o \, \overline{R}$ TEST DATA

TEST NO: DETROIT SEWER, WES 3-19, SILTY $K_o\overline{R}$ UNDRAINED LOADING STAGE

LINE NO	DEV STRESS TSF	AXIAL STRAIN %	INDUCED PWP TSF	EFFECT S 3 TSF	"A" PARAM	EFFECT S1/S3	S STRESS "Q" TSF	RSTRESS "P" TSF	RT CYL DEV STR TSF
3	1.235	0.	0.	0.828	0.	2.492	0.618	1.446	1.235
4	1.567	0.031	0.086	0.742	0.055	3.113	0.783	1.525	1.567
5	1.789	0.124	0.144	0.684	0.080	3.616	0.895	1.579	1.789
6	2.006	0.295	0.187	0.641	0.093	4.131	1.003	1.644	2.006
7	2.438	0.773	0.194	0.634	0.080	4.848	1.219	1.853	2.438
8	2.648	1.025	0.144	0.684	0.054	4.872	1.324	2.008	2.648
9	2.862	1.287	0.086	0.742	0.030	4.857	1.431	2.172	2.862
10	2.965	1.537	-0.029	0.857	-0.010	4.460	1.482	2.339	2.965
11	3.057	1.956	-0.331	1.159	-0.108	3.639	1.529	2.689	3.057
12	3.166	2.111	-0.396	1.224	-0.125	3.586	1.583	2.807	3.166
13	3.265	2.174	-0.410	1.238	-0.126	3.638	1.632	2.871	3.265
14	3.372	2.406	-0.504	1.332	-0.149	3.531	1.686	3.018	3.372
15	3.460	2.453	-0.518	1.346	-0.149	3.584	1.730	3.086	3.460
16	3.472	2.500	-0.518	1.346	-0.149	3.579	1.736	3.083	3.472
17	3.585	2.624	-0.590	1.418	-0.165	3.527	1.792	3.211	3.585
18	3.680	2.670	-0.598	1.426	-0.162	3.582	1.840	3.266	3.680
19	3.674	2.734	-0.670	1.498	-0.182	3.453	1.837	3.334	3.674
20	3.784	2.872	-0.684	1.512	-0.181	3.502	1.892	3.404	3.784
21	3.877	2.965	-0.698	1.526	-0.180	3.540	1.938	3.465	3.877
22	3.991	3.012	-0.749	1.577	-0.188	3.531	1.996	3.572	3.991
23	4.088	3.183	-0.792	1.620	-0.194	3.523	2.044	3.664	4.088

End of K_o consol.

TEST NO: DETROIT SEWER, WES 3-24, SILTY $K_o \overline{R}$ UNDRAINED LOADING STAGE

LINE NO	DEV STRESS TSF	AXIAL STRAIN %	INDUCED PWP TSF	EFFECT S 3 TSF	"A" PARAM	EFFECT SI/S3	SSTRESS "Q" TSF	NSTRESS "P" TSF	RT CYL DEV STR TSF
3	1.289	0.	0.	1.037	0.	2.243	0.644	1.681	1.289
4	1.407	0.016	0.036	1.001	0.026	2.406	0.703	1.704	1.407
5	1.525	0.032	0.072	0.965	0.047	2.580	0.762	1.727	1.525
6	1.628	0.063	0.094	0.943	0.058	2.726	0.814	1.757	1.628
7	1.749	0.095	0.108	0.929	0.062	2.884	0.875	1.804	1.749
8	1.852	0.126	0.115	0.922	0.062	3.010	0.926	1.848	1.852
9	1.976	0.173	0.115	0.922	0.058	3.144	0.988	1.910	1.976
10	2.078	0.205	0.108	0.929	0.052	3.237	1.039	1.968	2.078
11	2.199	0.252	0.101	0.936	0.046	3.349	1.099	2.035	2.199
12	2.304	0.284	0.094	0.943	0.041	3.442	1.152	2.095	2.304
13	2.419	0.315	0.079	0.958	0.033	3.527	1.210	2.167	2.419
14	2.521	0.347	0.065	0.972	0.026	3.593	1.260	2.232	2.521
15	2.747	0.425	0.029	1.008	0.010	3.725	1.374	2.382	2.747
16	2.858	0.457	0.007	1.030	0.003	3.776	1.429	2.459	2.858
17	2.976	0.488	-0.007	1.044	-0.002	3.851	1.488	2.532	2.976
18	3.080	0.520	-0.029	1.066	-0.009	3.891	1.540	2.606	3.080
19	3.192	0.551	-0.050	1.087	-0.016	3.936	1.596	2.683	3.192
20	3.426	0.614	-0.101	1.138	-0.029	4.012	1.713	2.851	3.426
21	3.653	0.662	-0.151	1.188	-0.041	4.075	1.827	3.015	3.653
22	3.850	0.725	-0.194	1.231	-0.050	4.127	1.925	3.156	3.850
23	4.086	0.772	-0.245	1.282	-0.060	4.188	2.043	3.324	4.086
24	4.313	0.835	-0.302	1.339	-0.070	4.220	2.156	3.496	4.313
25	4.514	0.882	-0.346	1.382	-0.077	4.265	2.257	3.639	4.514
26	4.737	0.930	-0.396	1.433	-0.084	4.306	2.368	3.801	4.737
27	4.968	0.977	-0.446	1.483	-0.090	4.350	2.484	3.967	4.968
28	5.197	1.024	-0.504	1.541	-0.097	4.373	2.599	4.139	5.197
29	5.397	1.071	-0.547	1.584	-0.101	4.407	2.699	4.283	5.397

End of K_o consol.

[illegible]: DETROIT SEWER, WES 4-24, CLAYEY $K_o \overline{R}$ UNDRAINED LOADING STAGE

	DEV STRESS TSF	AXIAL STRAIN %	INDUCED PWP TSF	EFFECT S 3 TSF	"A" PARAM	EFFECT S1/S3	SSTRESS "Q" TSF	NSTRESS "P" TSF	RT CYL DEV STR TSF	
3	1.811	0.	0.	1.303	0.	2.390	0.906	2.209	1.811	End of K_o consol.
4	1.937	0.016	0.050	1.253	0.026	2.546	0.969	2.222	1.937	
5	2.044	0.080	0.029	1.274	0.014	2.604	1 022	2.297	2.044	
6	2.146	0.128	0.072	1.231	0.034	2.743	1.073	2.304	2.146	
7	2.261	0.304	0.065	1.238	0.029	2.826	1.130	2.369	2.261	
8	2.364	0.415	0.101	1.202	0.043	2.966	1.182	2.384	2.364	
9	2.468	0.623	0.094	1.210	0.038	3.041	1.234	2.444	2.468	
10	2.567	1.006	0.058	1.246	0.022	3.061	1.284	2.529	2.567	
11	2.671	1.214	0.072	1.231	0.027	3.169	1.335	2.567	2.671	
12	2.761	1.565	0.022	1.282	0.008	3.154	1.381	2.662	2.761	
13	2.862	1.805	0.	1.303	0.	3.196	1.431	2.734	2.862	
14	2.975	2.029	-0.029	1.332	-0.010	3.233	1.487	2.819	2.975	
15	3.050	2.668	-0.108	1.411	-0.035	3.161	1.525	2.936	3.050	
16	3.145	2.891	-0.137	1.440	-0.043	3.184	1.573	3.013	3.145	
17	3.234	3.435	-0.202	1.505	-0.062	3.149	1.617	3.122	3.234	
18	3.343	3.482	-0.202	1.505	-0.060	3.222	1.672	3.176	3.343	
19	3.443	3.610	-0.238	1.541	-0.069	3.234	1.721	3.262	3.443	
20	3.602	4.888	-0.389	1.692	-0.108	3.129	1.801	3.493	3.602	
21	3.700	5.048	-0.418	1.721	-0.113	3.150	1.850	3.571	3.700	
22	3.787	5.335	-0.468	1.771	-0.124	3.138	1.893	3.664	3.787	

TEST NO: DETROIT SEWER, WES 4-30, SILTY K_o $\overline{R}$ UNDRAINED LOADING STAGE

LINE NO	DEV STRESS TSF	AXIAL STRAIN %	INDUCED PWP TSF	EFFECT S 3 TSF	"A" PARAM	EFFECT S1/S3	SSTRESS "Q" TSF	NSTRESS "P" TSF	RT CYL DEV STR TSF	
3	2.071	0.	0.	1.505	0.	2.377	1.036	2.540	2.071	← End of K_o consol.
4	2.110	0.016	0.014	1.490	0.007	2.416	1.055	2.545	2.110	
5	2.214	0.032	0.050	1.454	0.023	2.522	1.107	2.561	2.214	
6	2.322	0.048	0.079	1.426	0.034	2.629	1.161	2.586	2.322	
7	2.429	0.064	0.108	1.397	0.044	2.739	1.215	2.611	2.429	
8	2.538	0.096	0.137	1.368	0.054	2.855	1.269	2.637	2.538	
9	2.645	0.129	0.151	1.354	0.057	2.954	1.323	2.676	2.645	
10	2.751	0.161	0.166	1.339	0.060	3.054	1.375	2.715	2.751	
11	2.862	0.193	0.173	1.332	0.060	3.149	1.431	2.763	2.862	
12	2.962	0.241	0.173	1.332	0.058	3.224	1.481	2.813	2.962	
13	3.077	0.273	0.173	1.332	0.056	3.310	1.539	2.871	3.077	
14	3.183	0.321	0.166	1.339	0.052	3.377	1.592	2.931	3.183	
15	3.282	0.353	0.158	1.346	0.048	3.438	1.641	2.987	3.282	
16	3.395	0.402	0.144	1.361	0.042	3.495	1.698	3.059	3.395	
17	3.499	0.434	0.137	1.368	0.039	3.557	1.749	3.117	3.499	
18	3.599	0.466	0.122	1.382	0.034	3.604	1.800	3.182	3.599	
19	3.718	0.514	0.108	1.397	0.029	3.662	1.859	3.256	3.718	
20	3.813	0.546	0.094	1.411	0.025	3.702	1.907	3.318	3.813	
21	3.921	0.578	0.079	1.426	0.020	3.751	1.961	3.386	3.921	
22	4.021	0.610	0.058	1.447	0.014	3.778	2.011	3.458	4.021	
23	4.134	0.643	0.043	1.462	0.010	3.829	2.067	3.529	4.134	
24	4.248	0.675	0.022	1.483	0.005	3.864	2.124	3.607	4.248	
25	4.349	0.707	0.	1.505	0.	3.890	2.175	3.679	4.349	
26	4.446	0.739	-0.022	1.526	-0.005	3.913	2.223	3.750	4.446	
27	4.563	0.771	-0.043	1.548	-0.009	3.947	2.281	3.829	4.563	
28	4.606	0.787	-0.050	1.555	-0.011	3.962	2.303	3.858	4.606	
29	4.696	0.803	-0.065	1.570	-0.014	3.992	2.348	3.918	4.696	
30	4.774	0.819	-0.079	1.584	-0.017	4.014	2.387	3.971	4.774	
31	4.873	0.851	-0.101	1.606	-0.021	4.035	2.437	4.042	4.873	
32	4.987	0.884	-0.122	1.627	-0.025	4.065	2.494	4.121	4.987	
33	5.084	0.900	-0.144	1.649	-0.028	4.083	2.542	4.191	5.084	
34	5.190	0.932	-0.173	1.678	-0.033	4.094	2.595	4.273	5.190	
35	5.290	0.964	-0.194	1.699	-0.037	4.113	2.645	4.344	5.290	
36	5.397	0.996	-0.216	1.721	-0.040	4.136	2.699	4.419	5.397	
37	5.501	1.012	-0.245	1.750	-0.045	4.144	2.750	4.500	5.501	
38	5.620	1.044	-0.266	1.771	-0.047	4.173	2.810	4.581	5.620	
39	5.719	1.076	-0.288	1.793	-0.050	4.190	2.860	4.652	5.719	

APPENDIX F

OVALLING MEASUREMENTS

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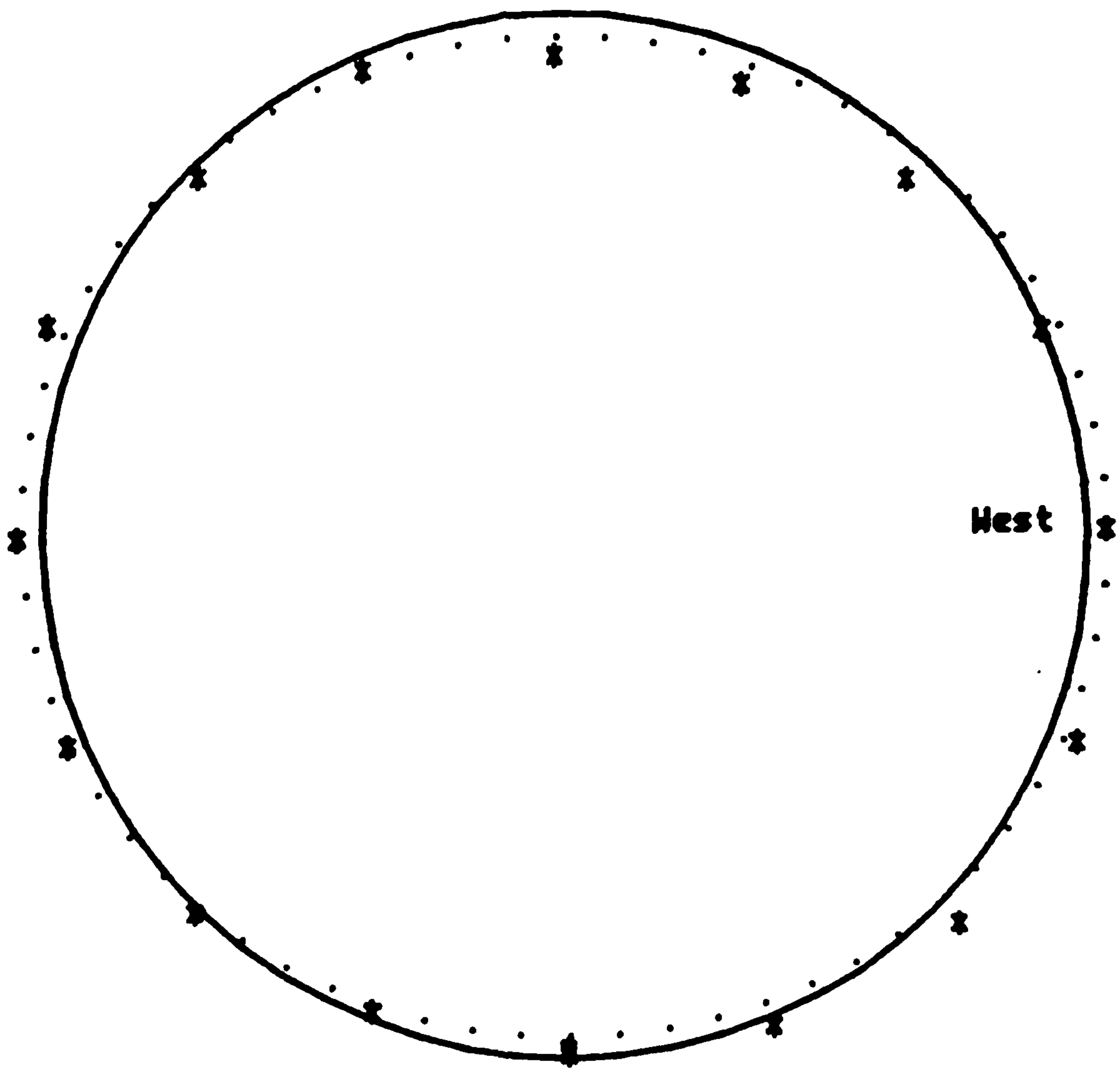

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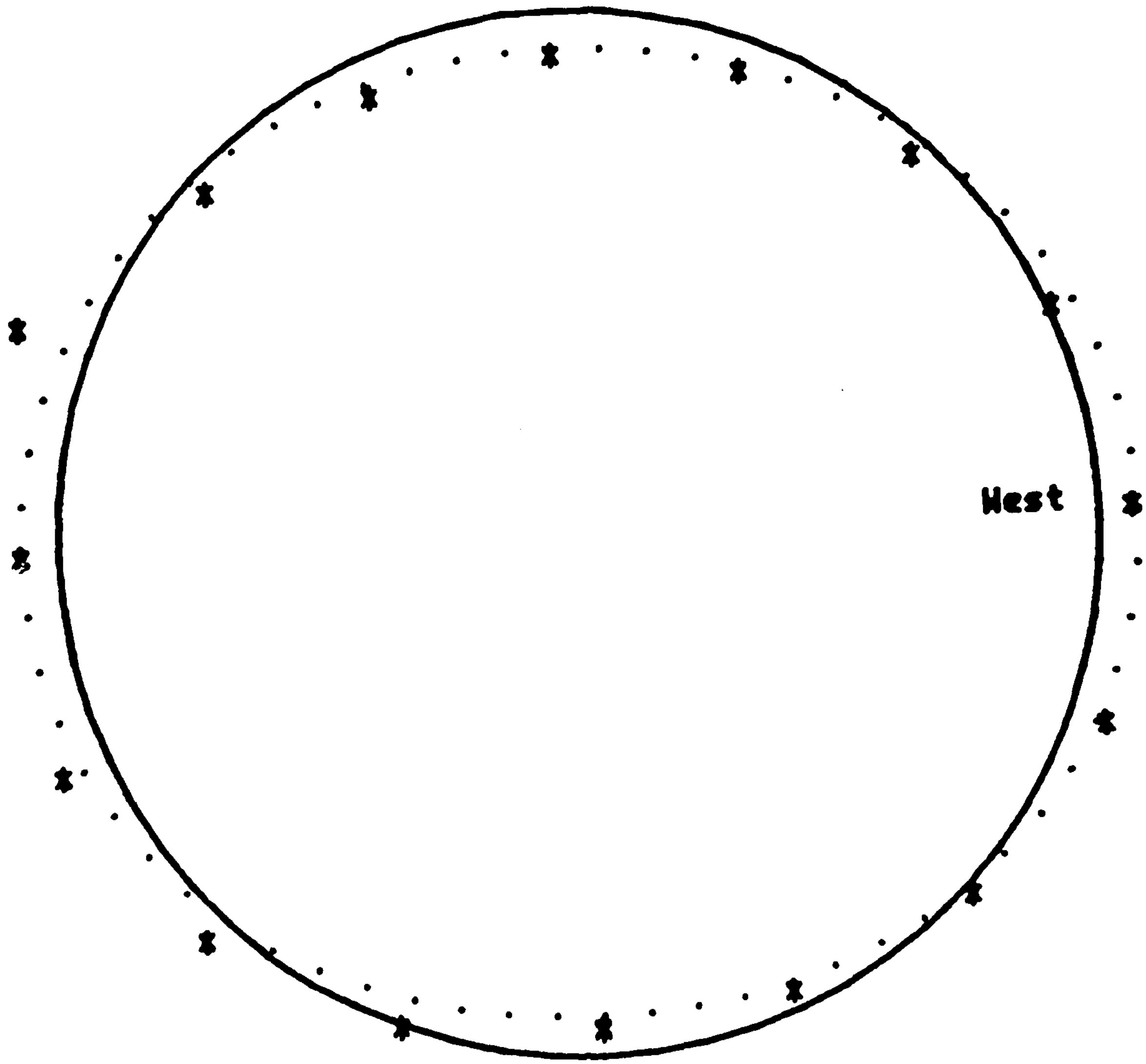

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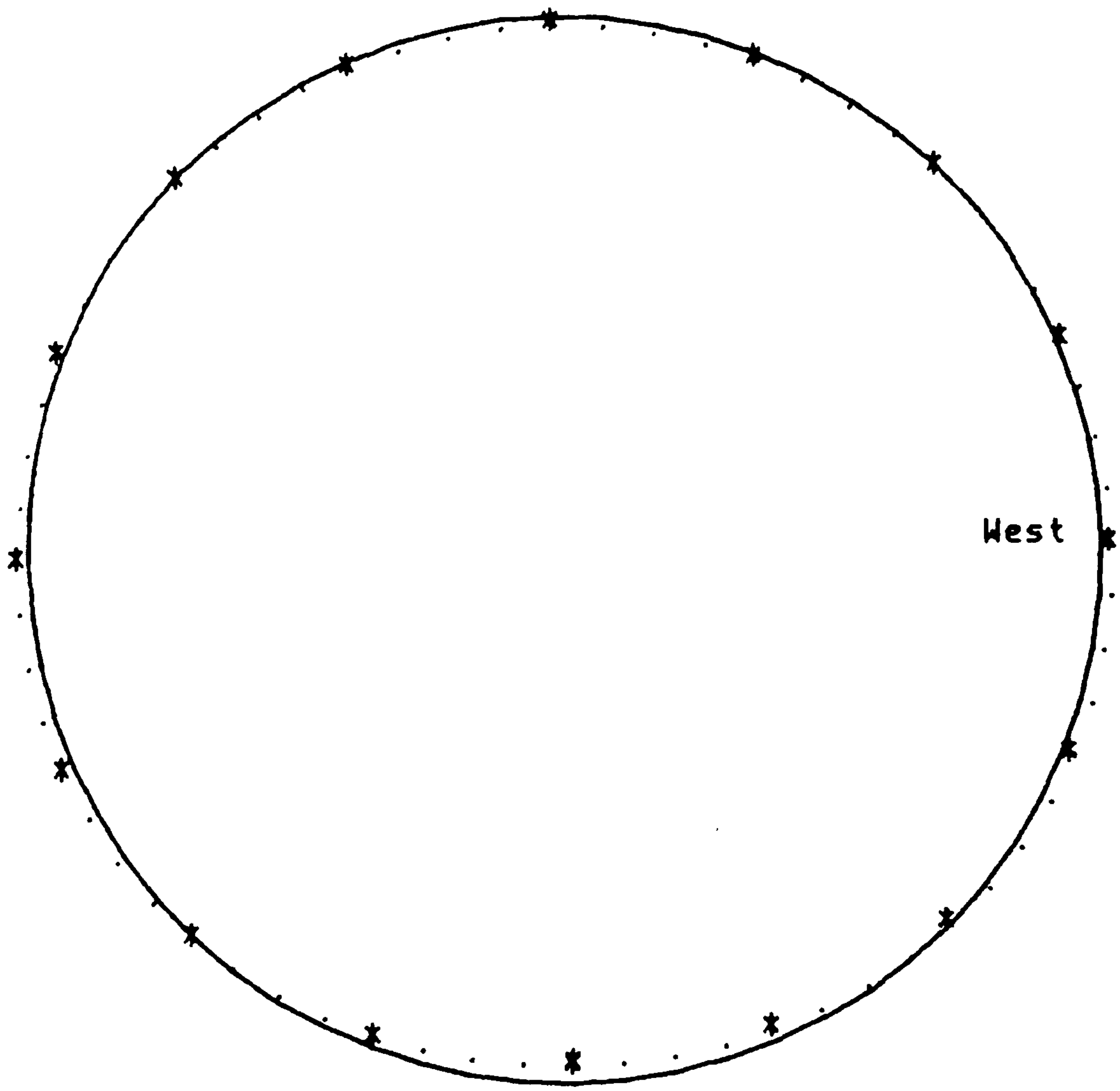

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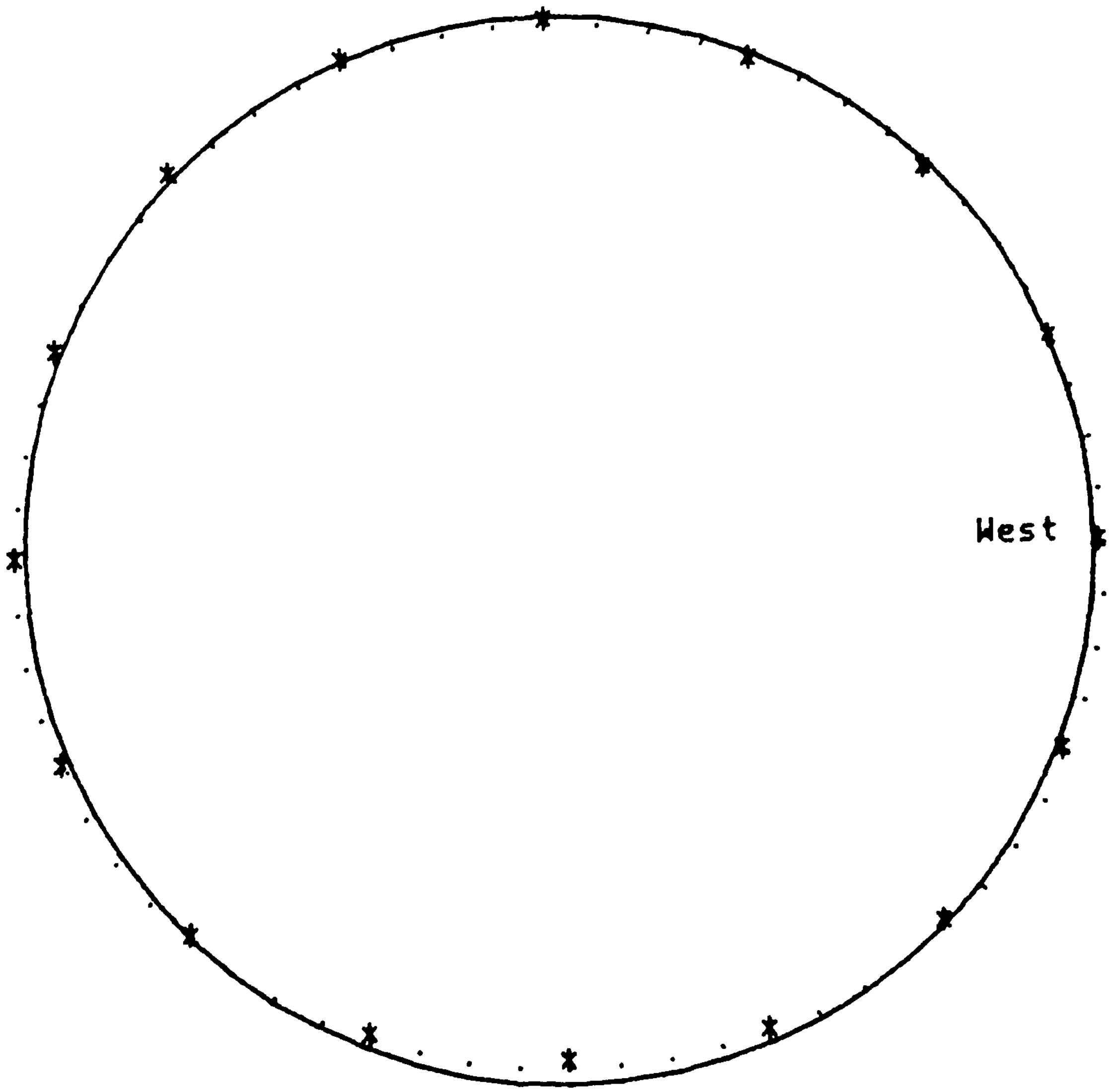

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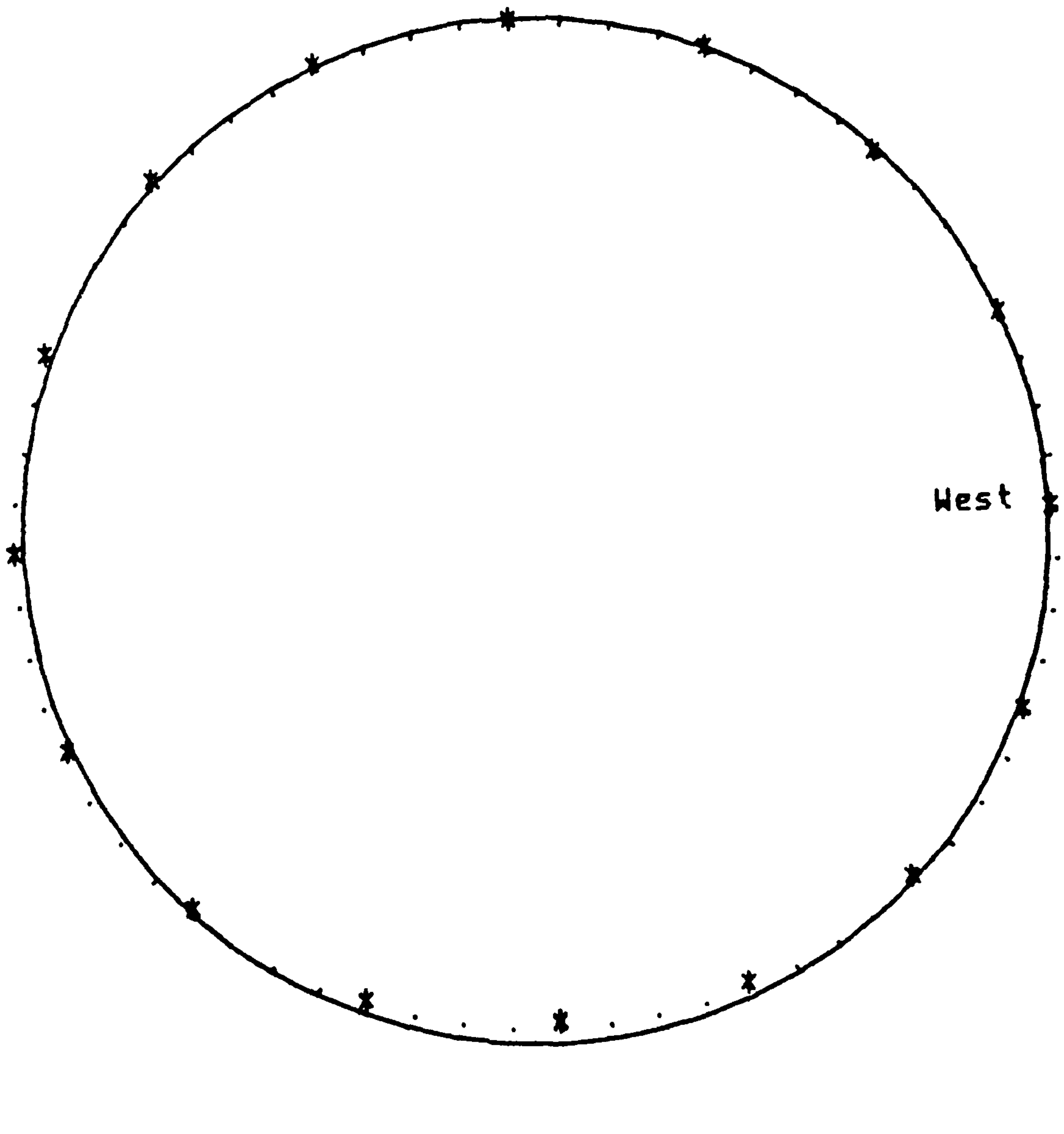

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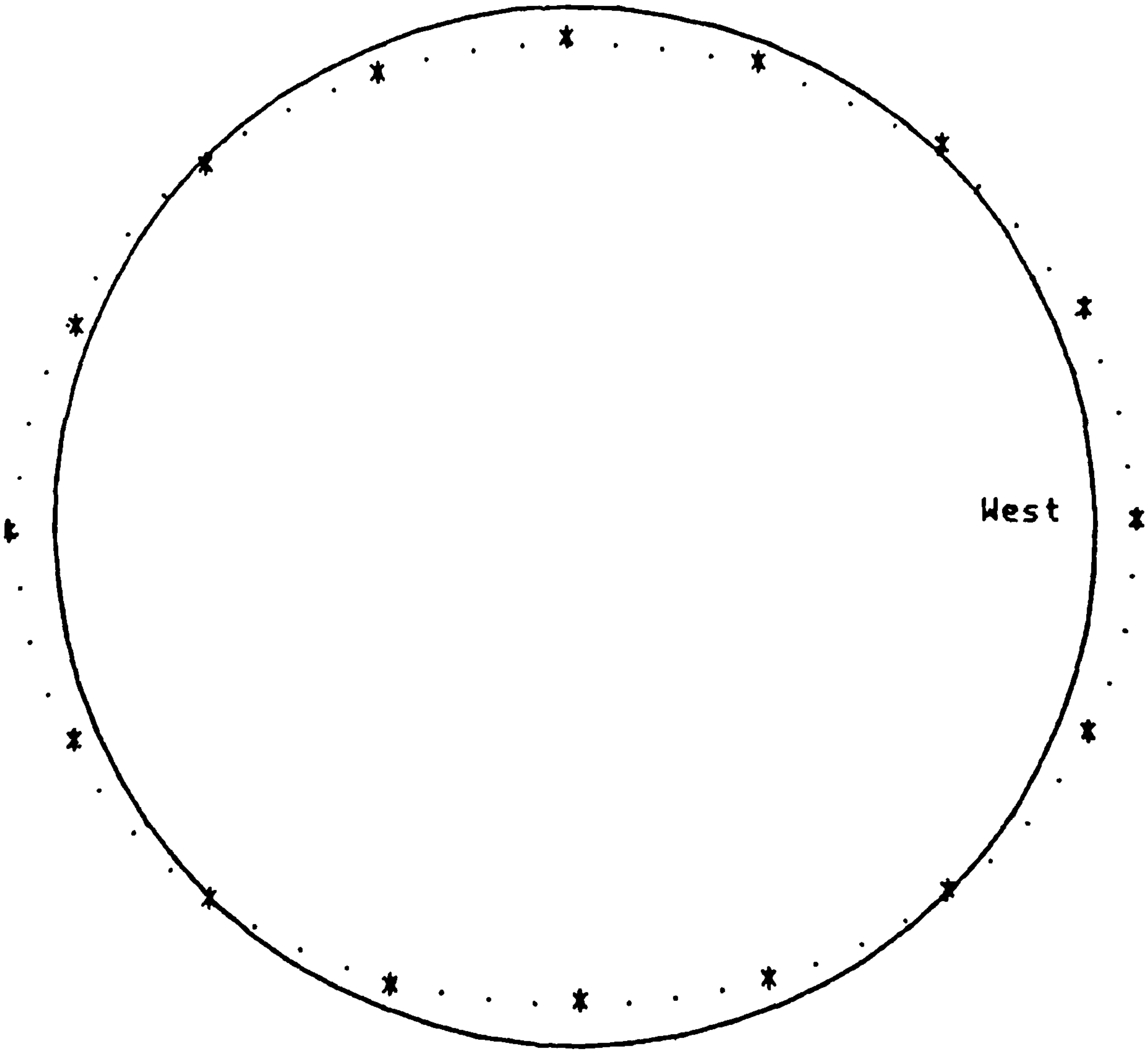

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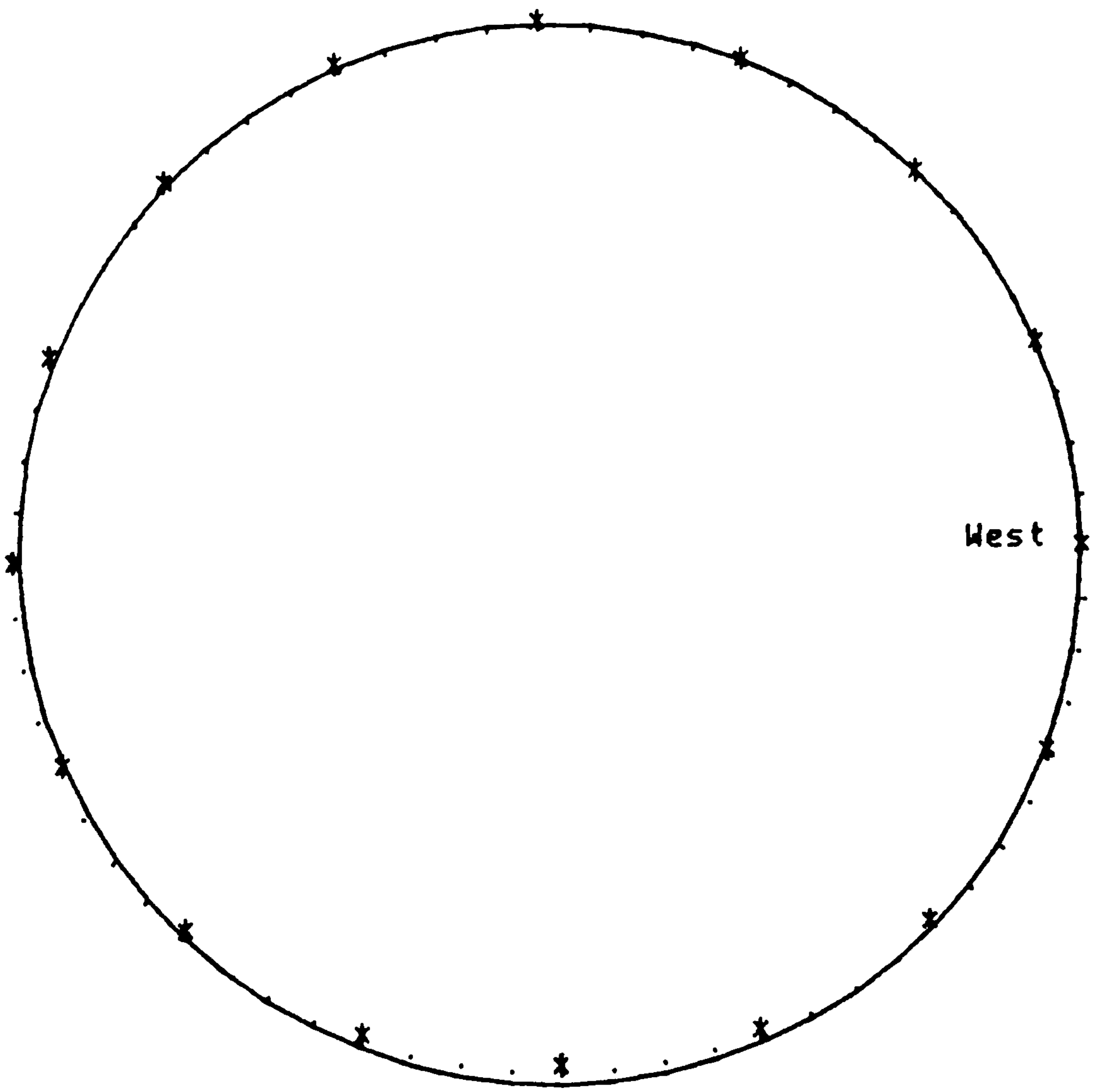

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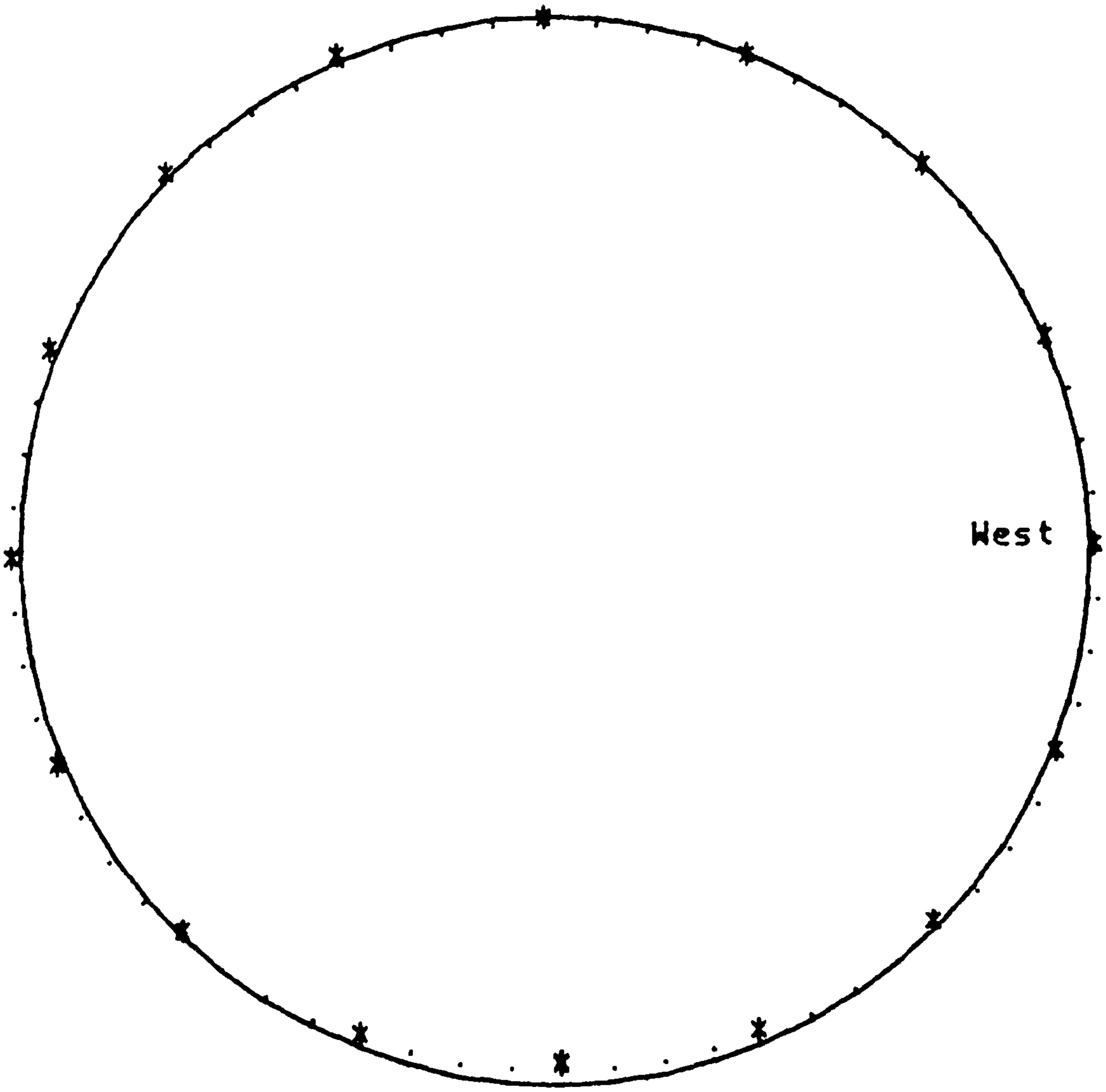

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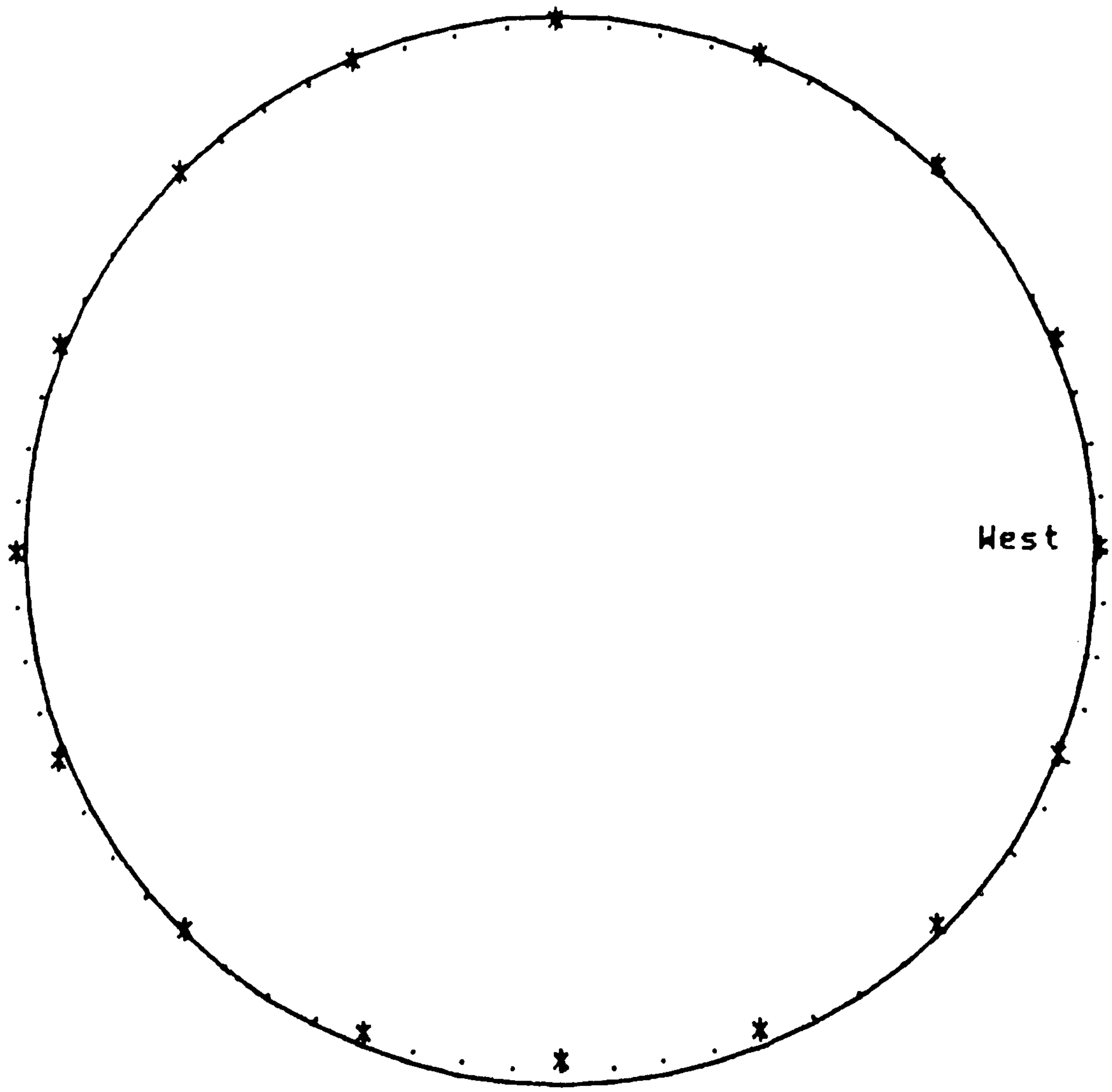

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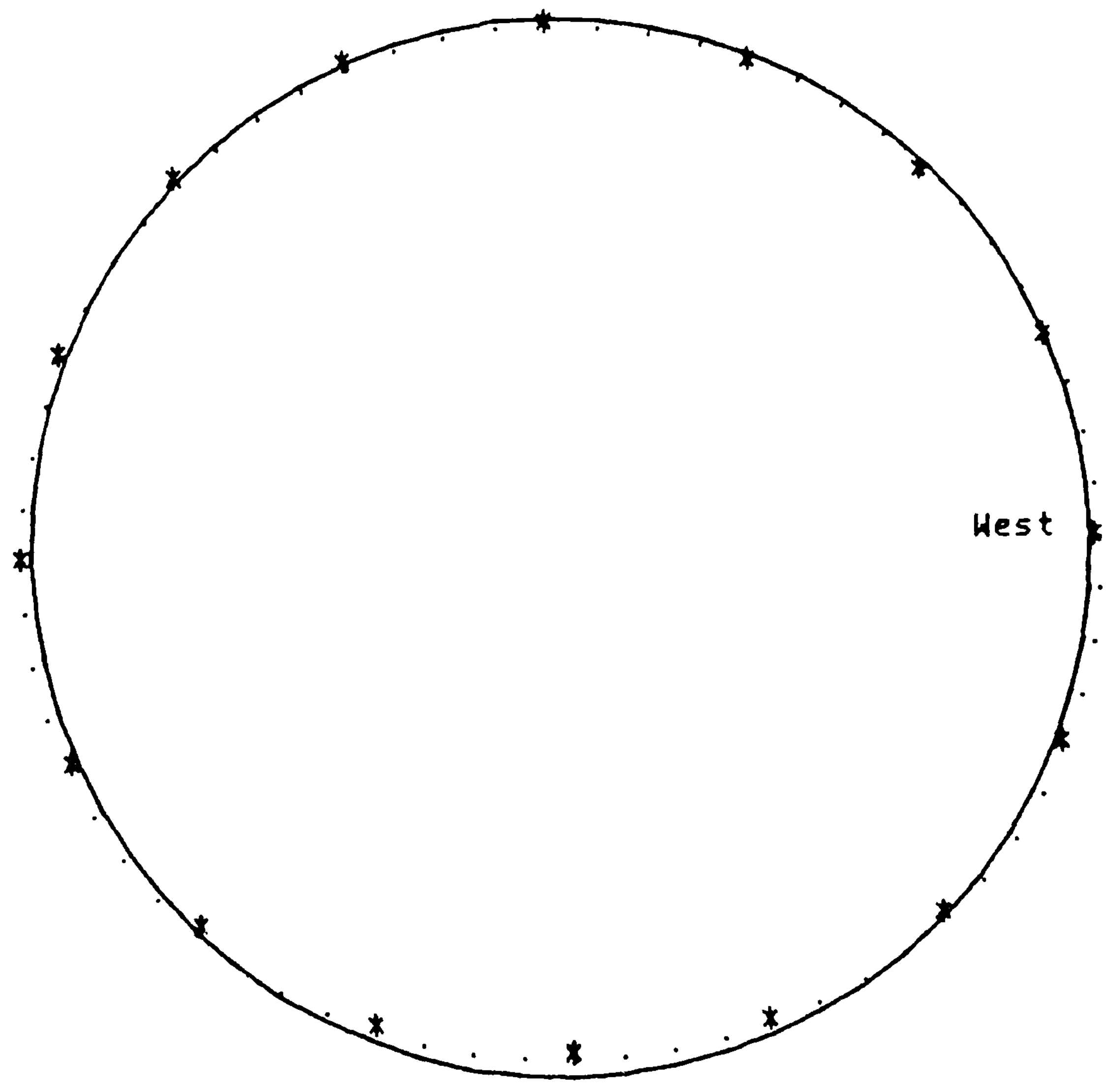

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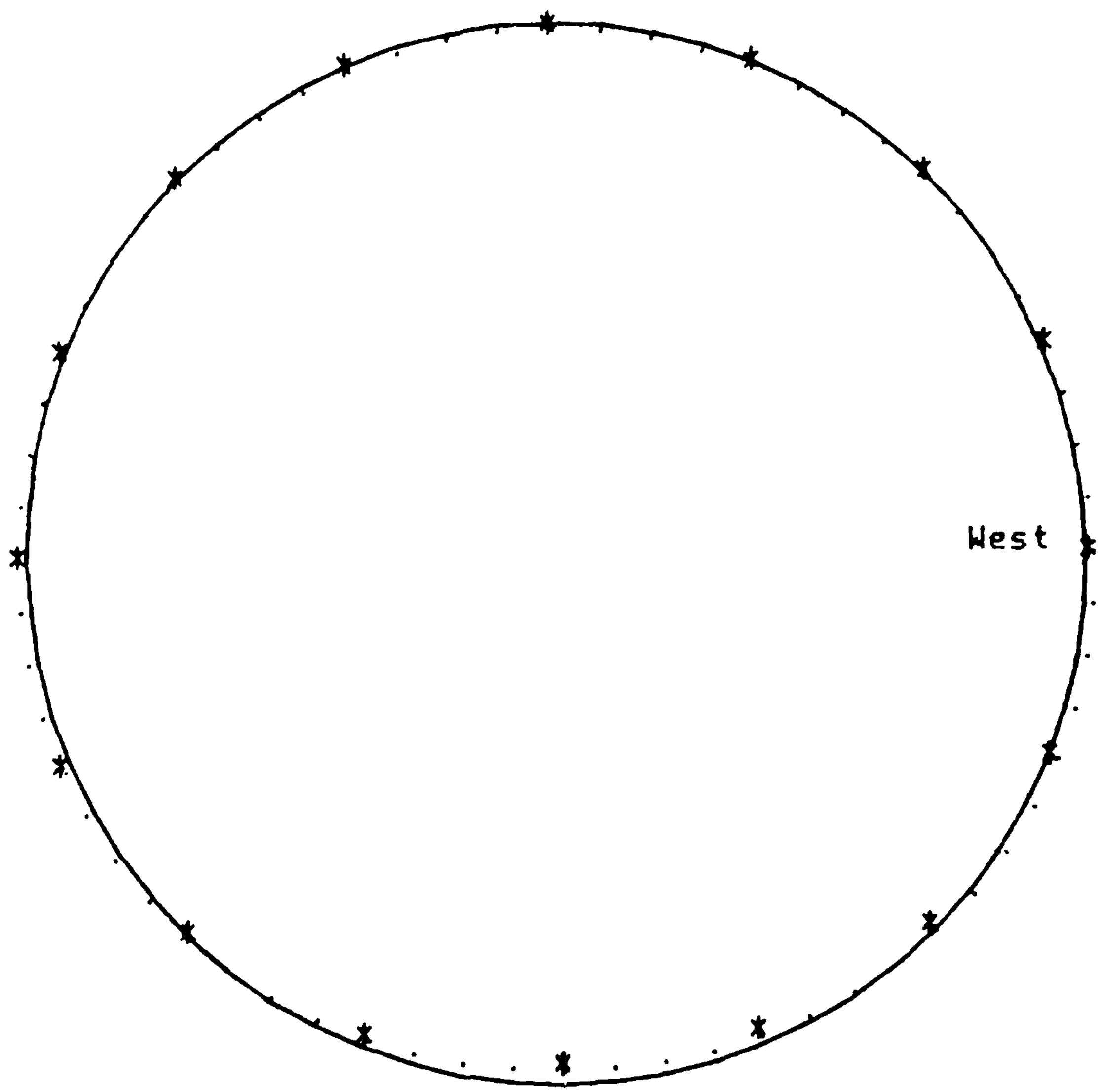

AD-A096 440   ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC   F/G 13/2
15 MILE ROAD/EDISON CORRIDOR SEWER TUNNEL FAILURE STUDY, DETROI--ETC(U)
JAN 81   D ALBERT, G C HOFF, B LORENCE   NCE-IA-80-055
UNCLASSIFIED   WES/TR/GL-81-2   NL

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PCI-7 STA 115+00

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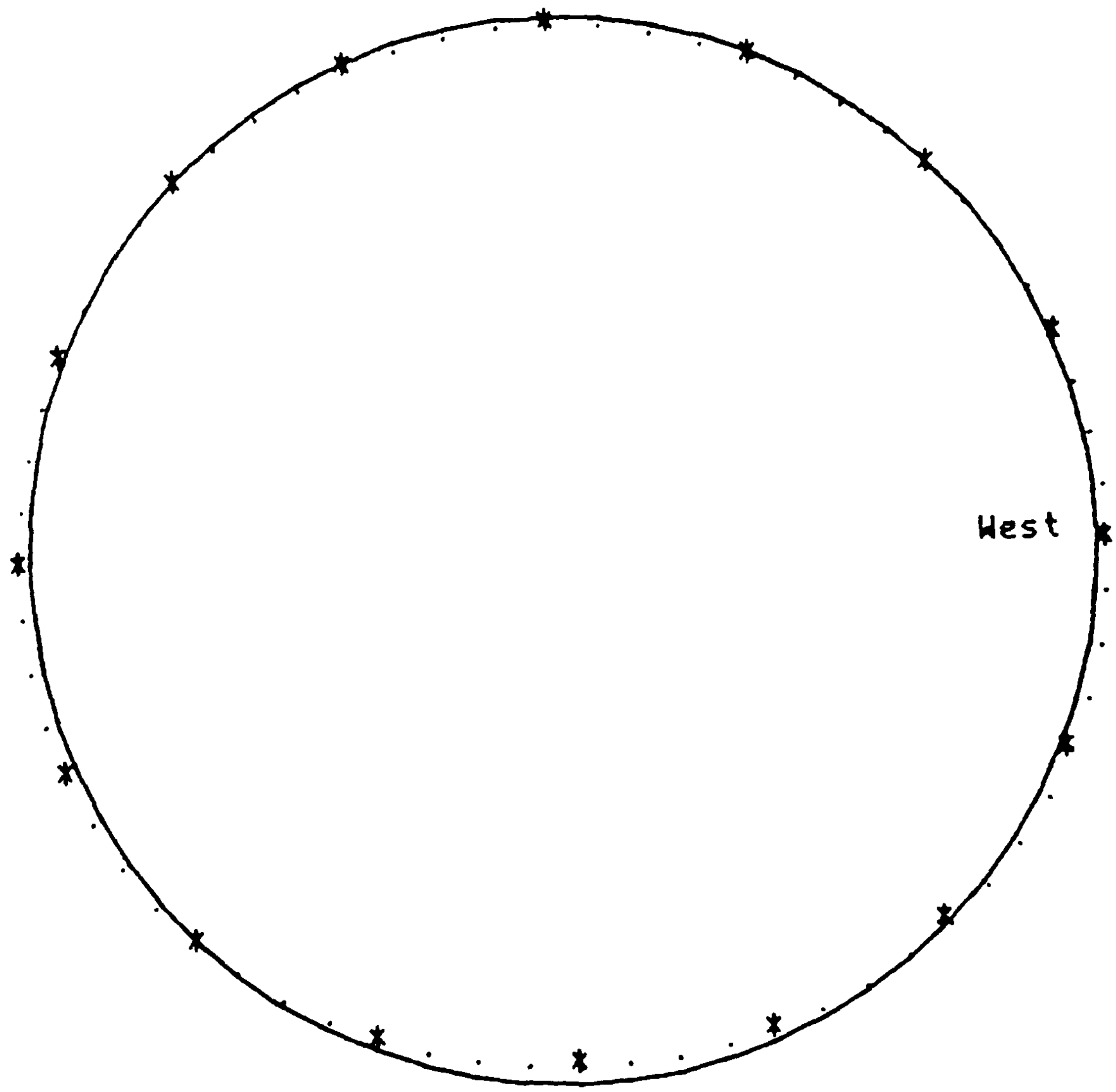

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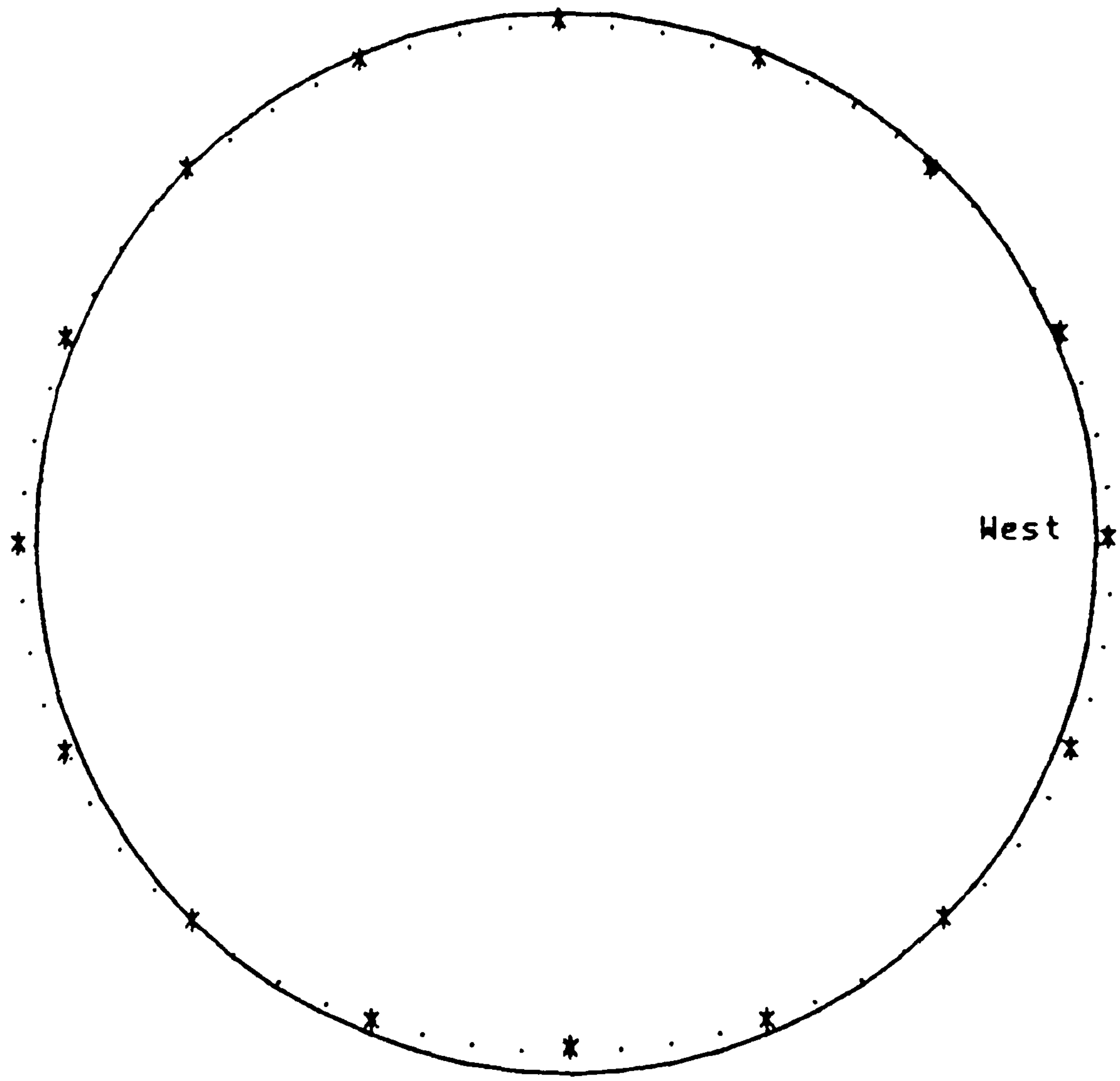

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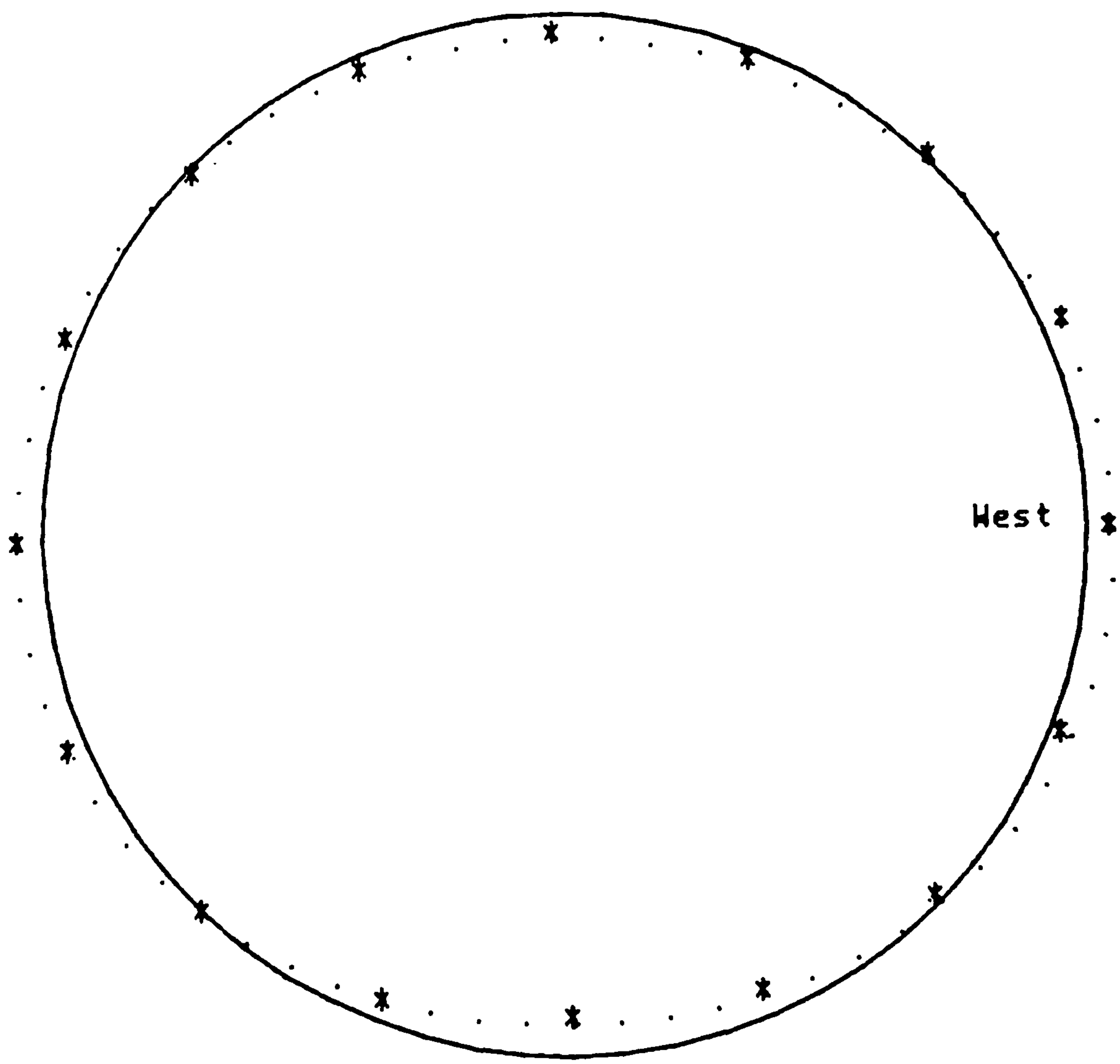

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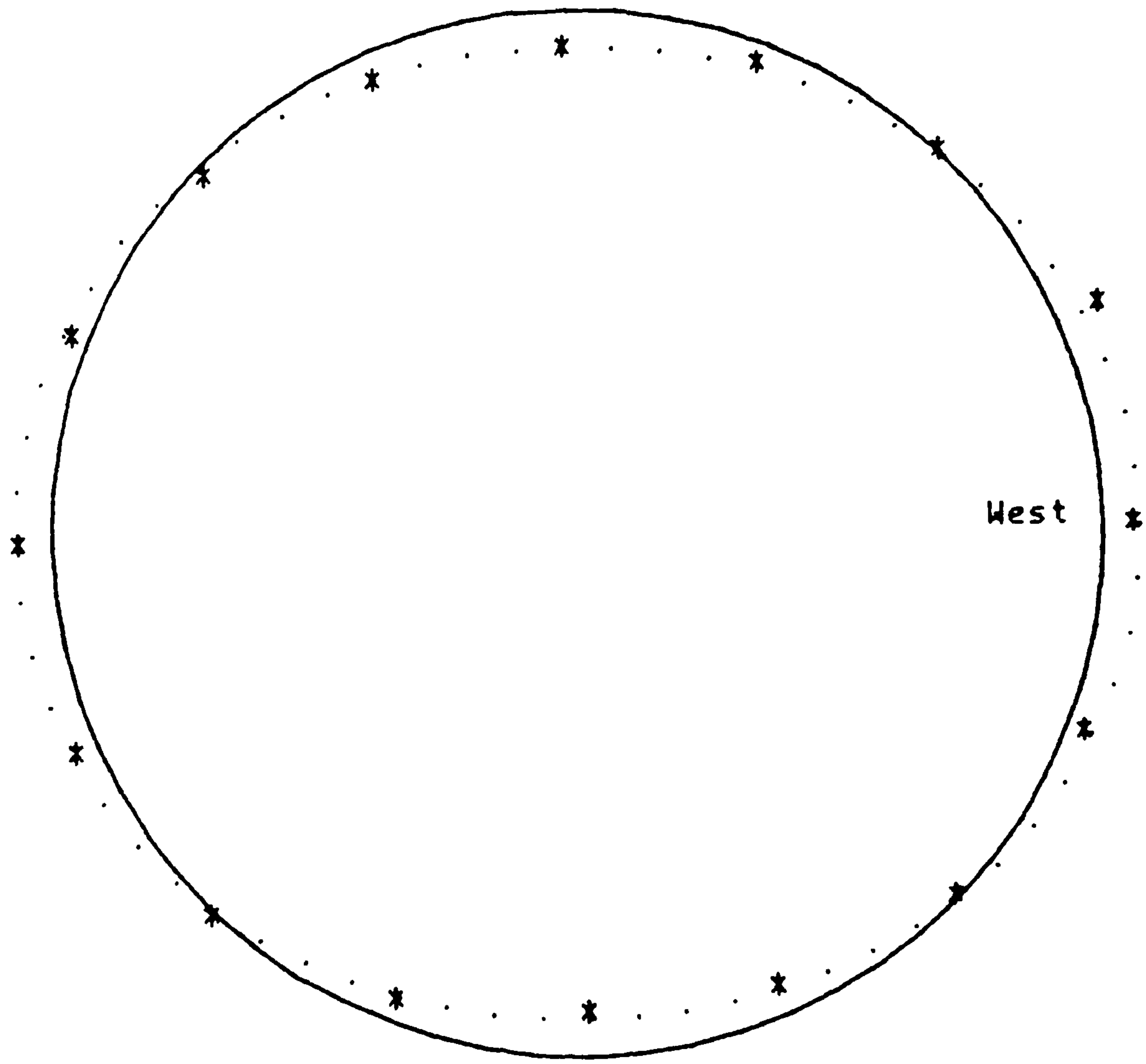

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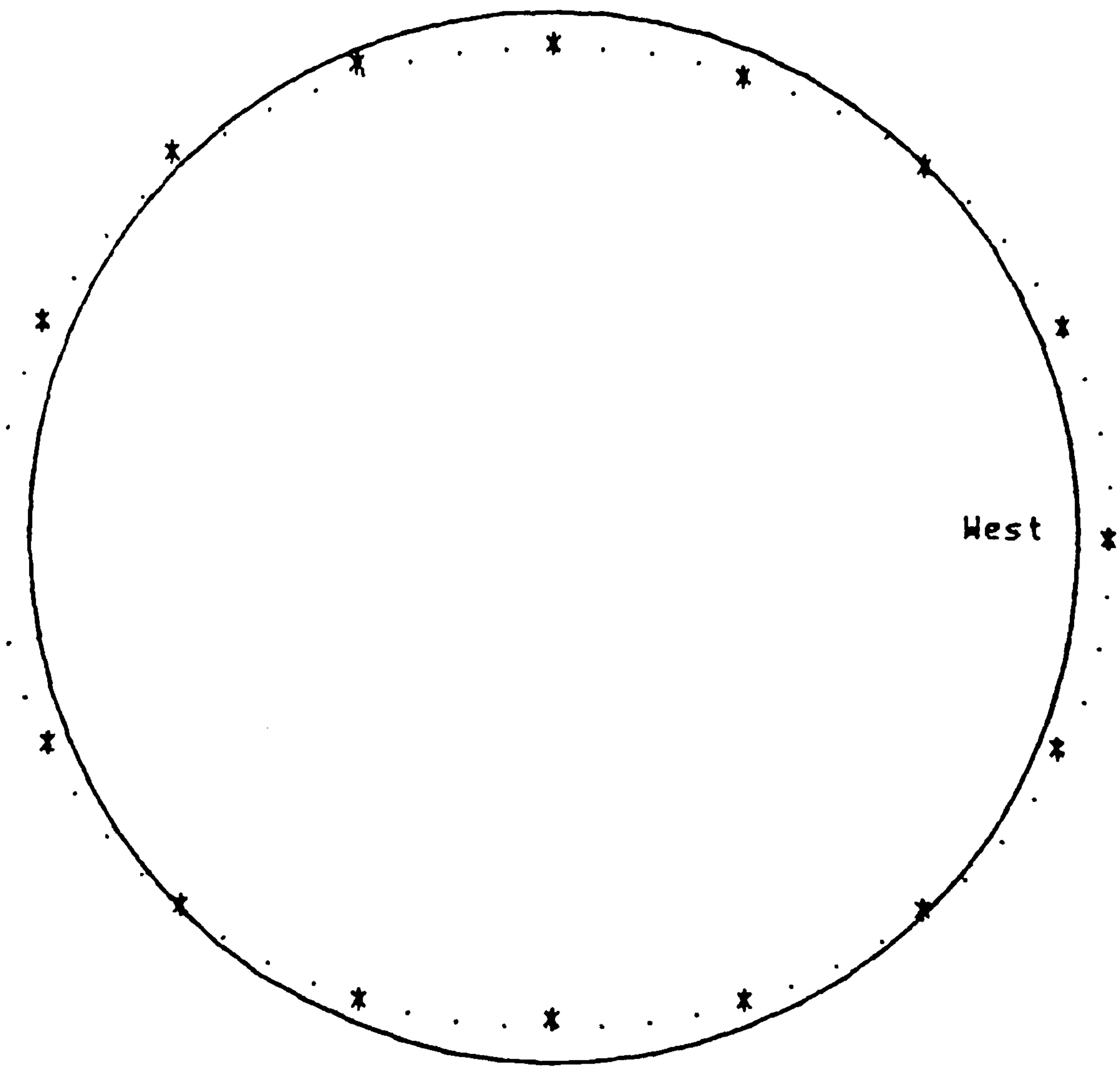

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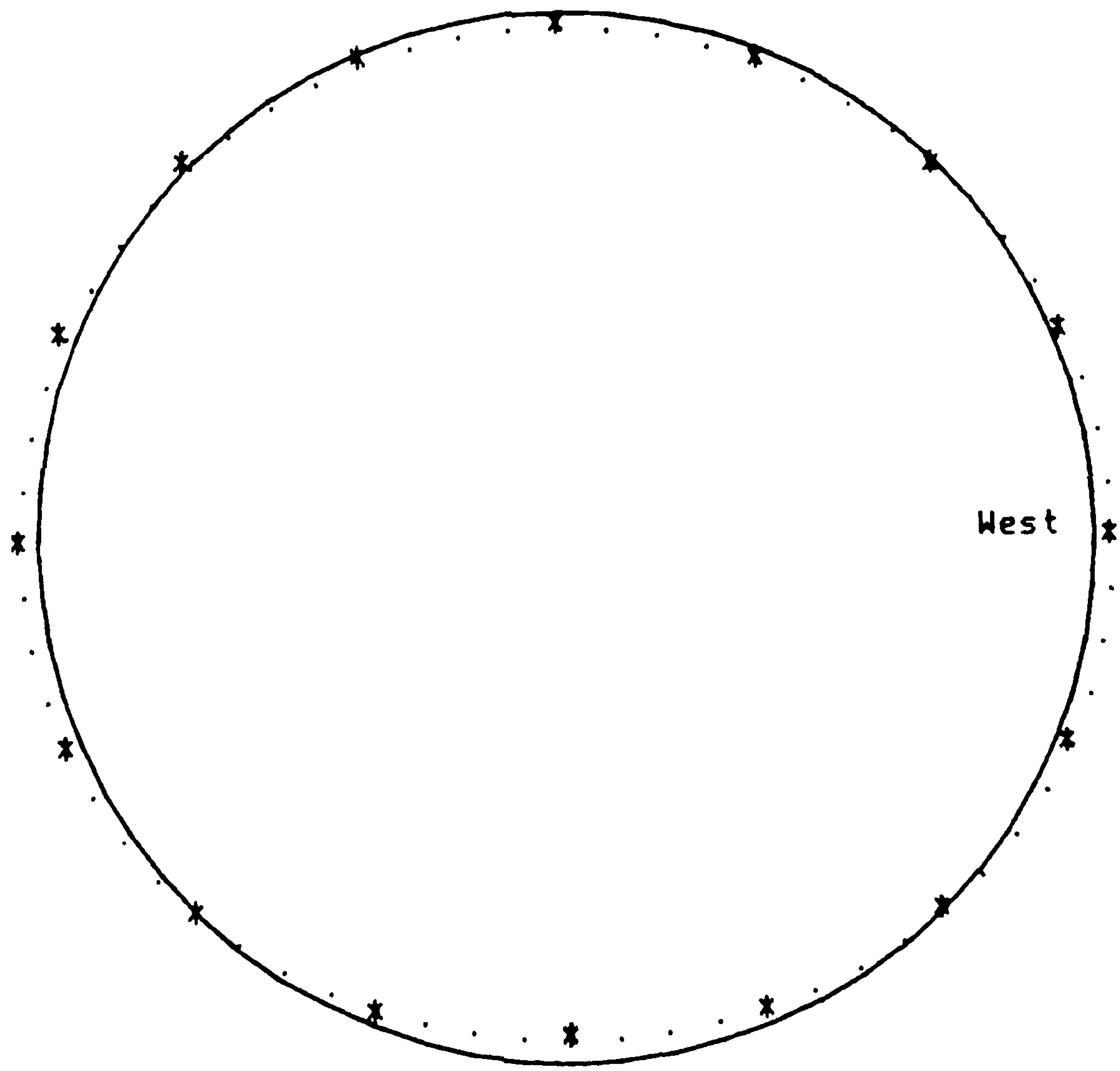

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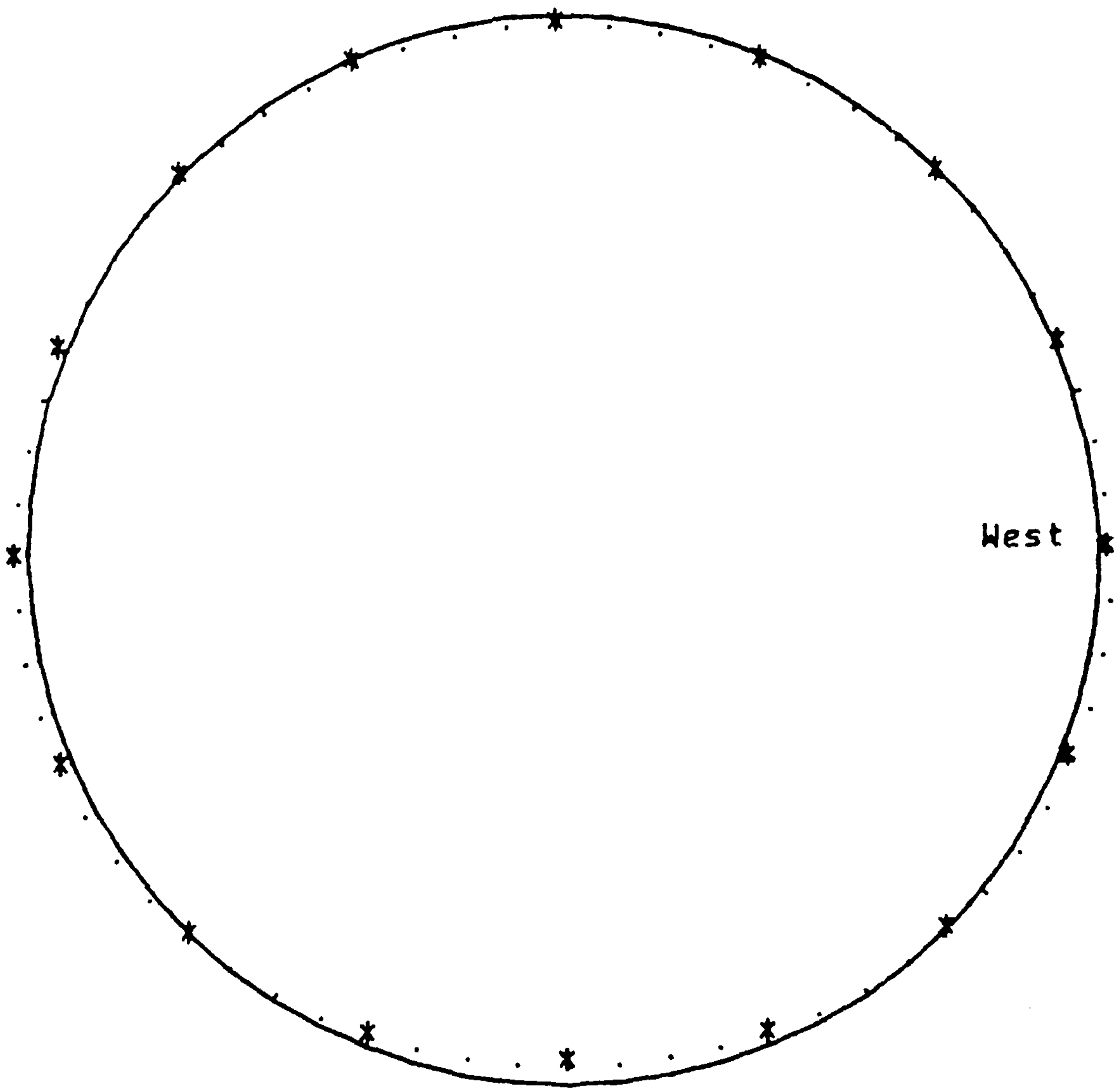

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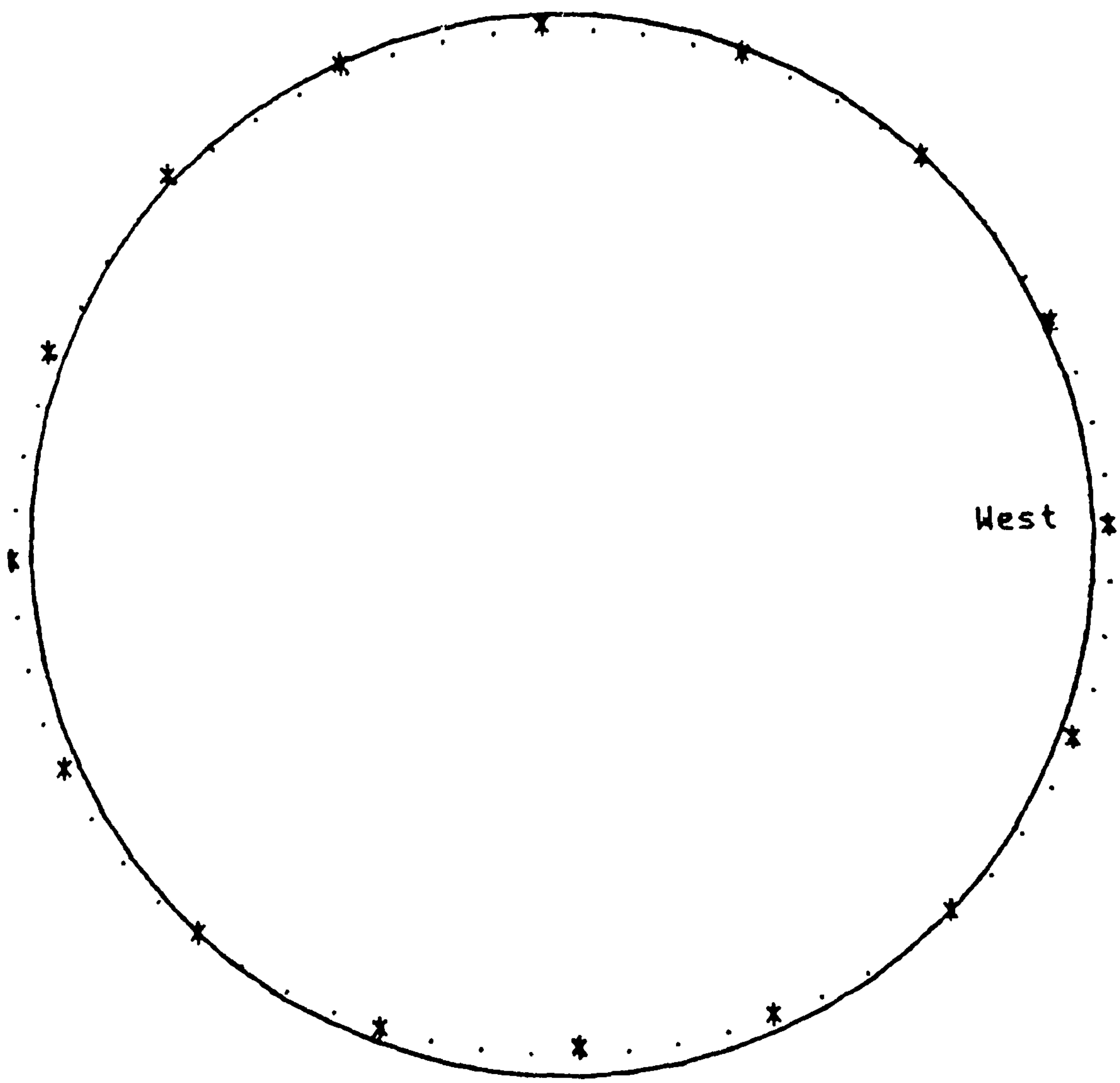

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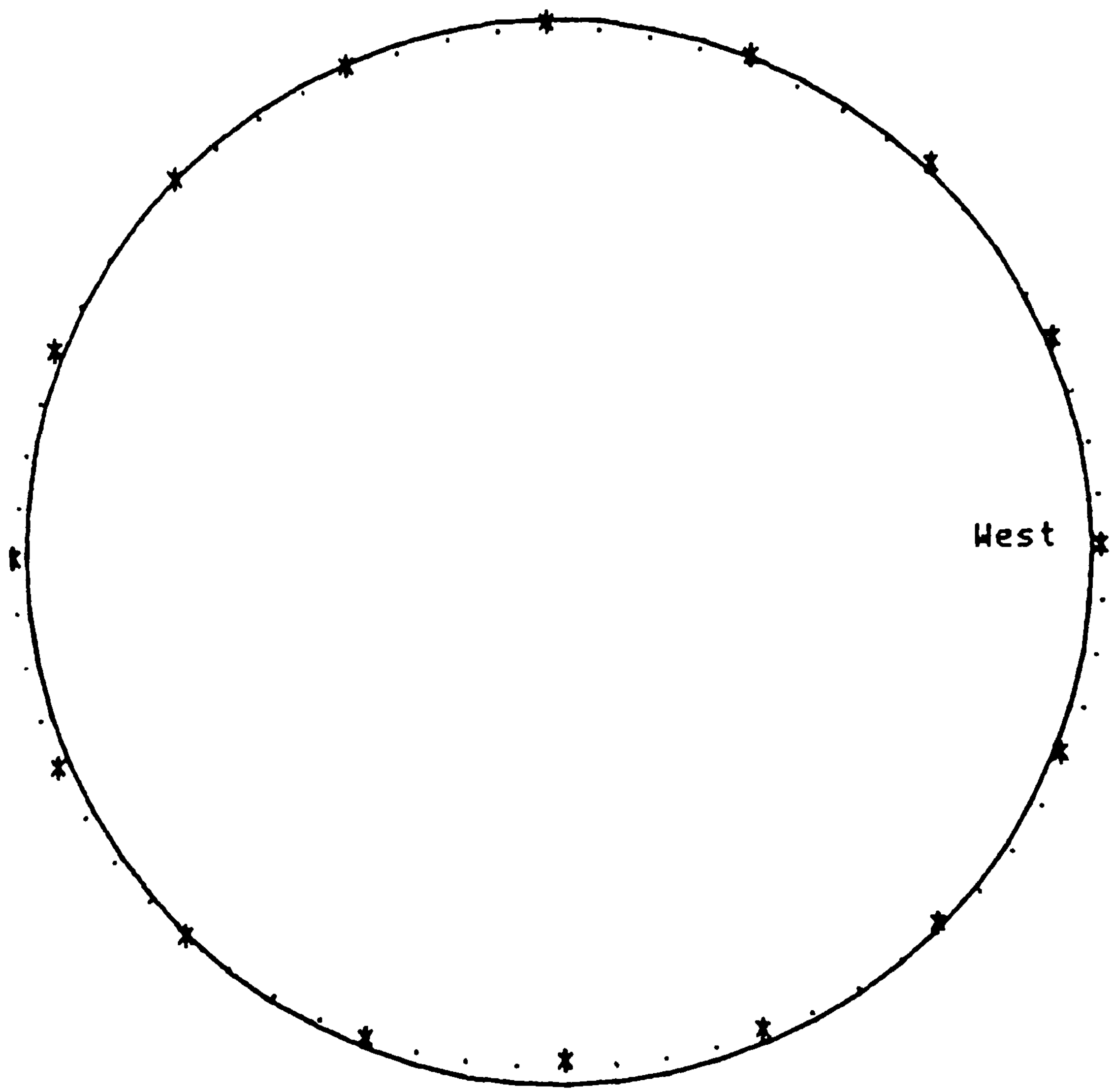

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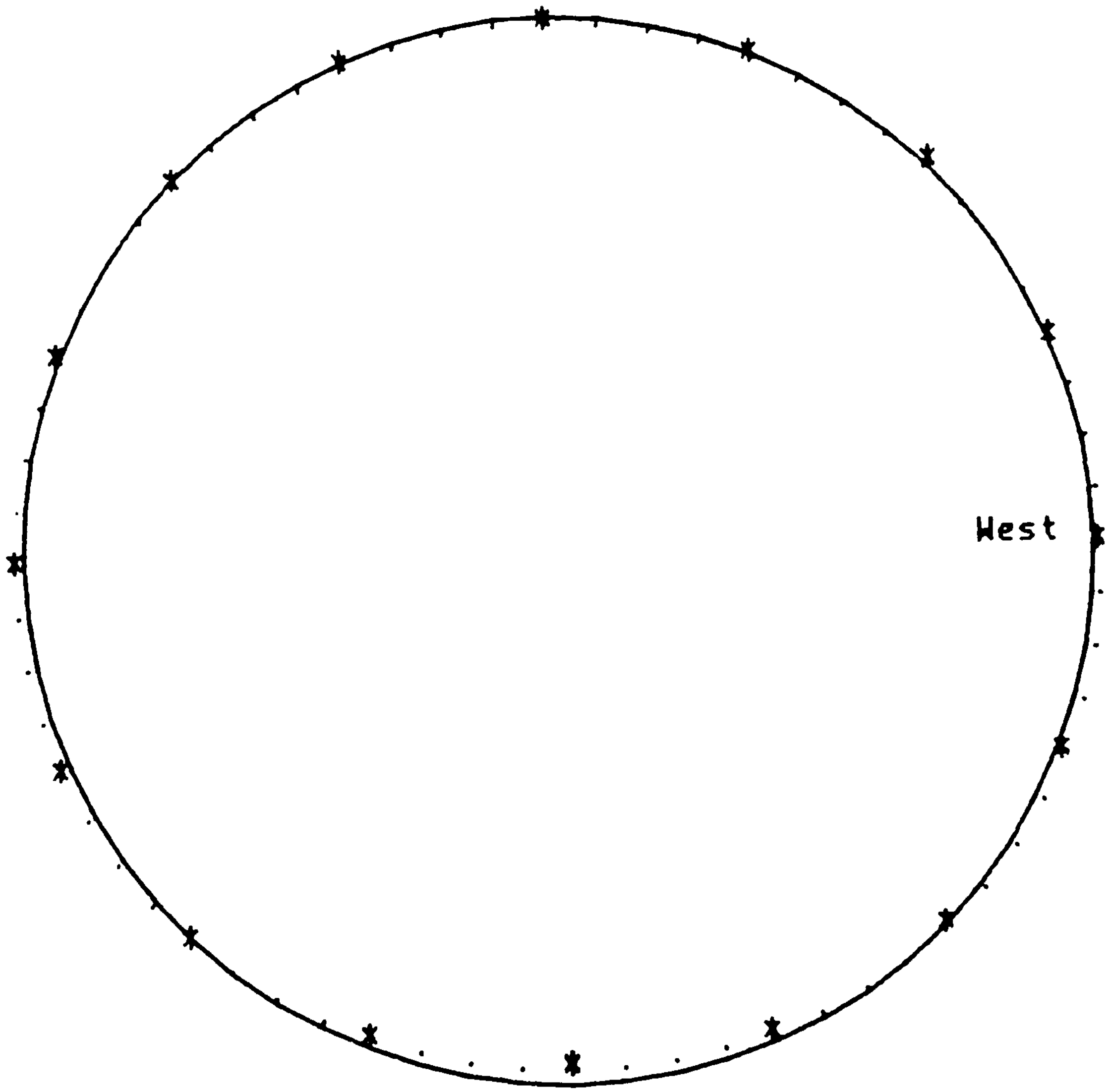

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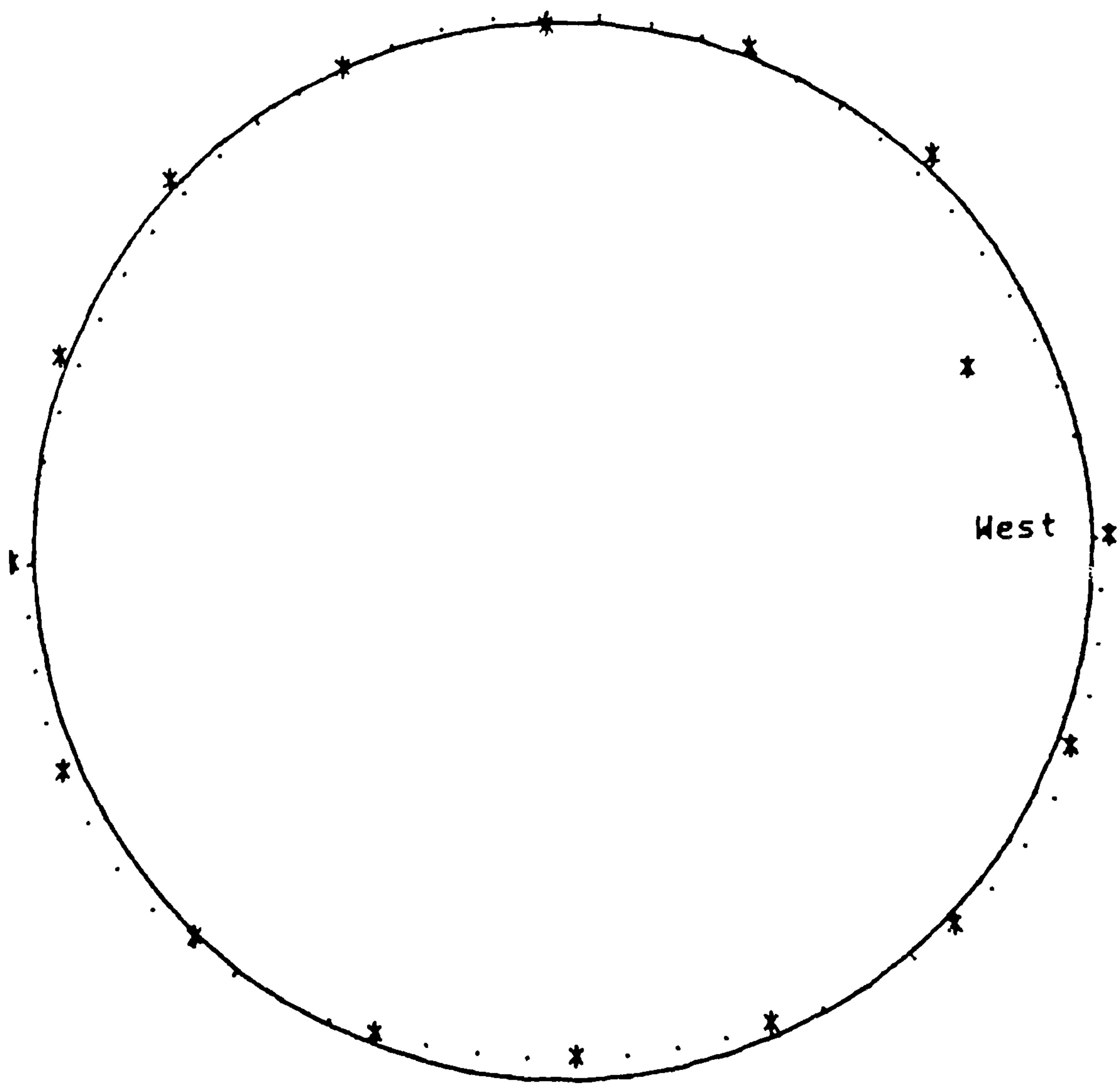

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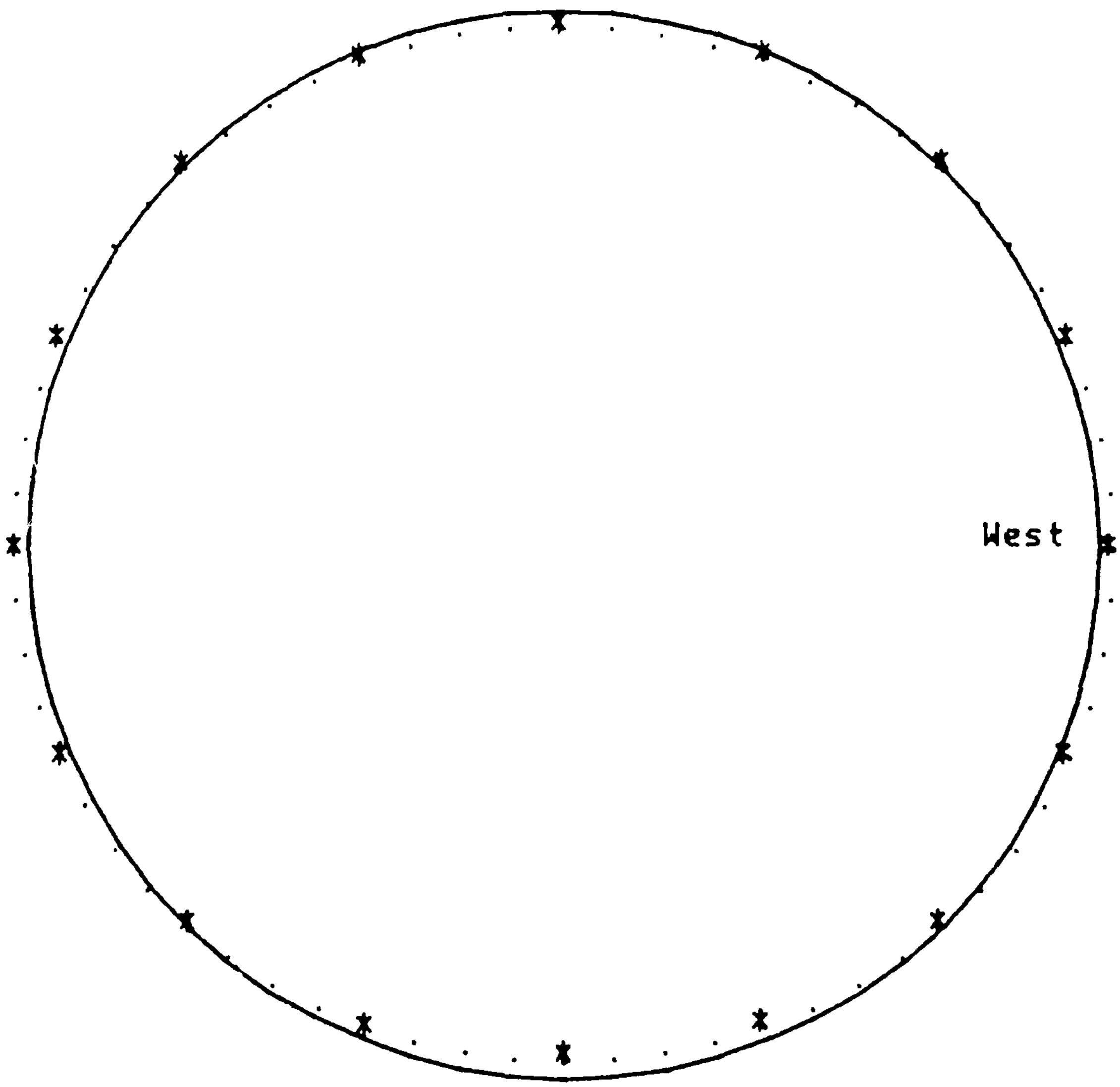

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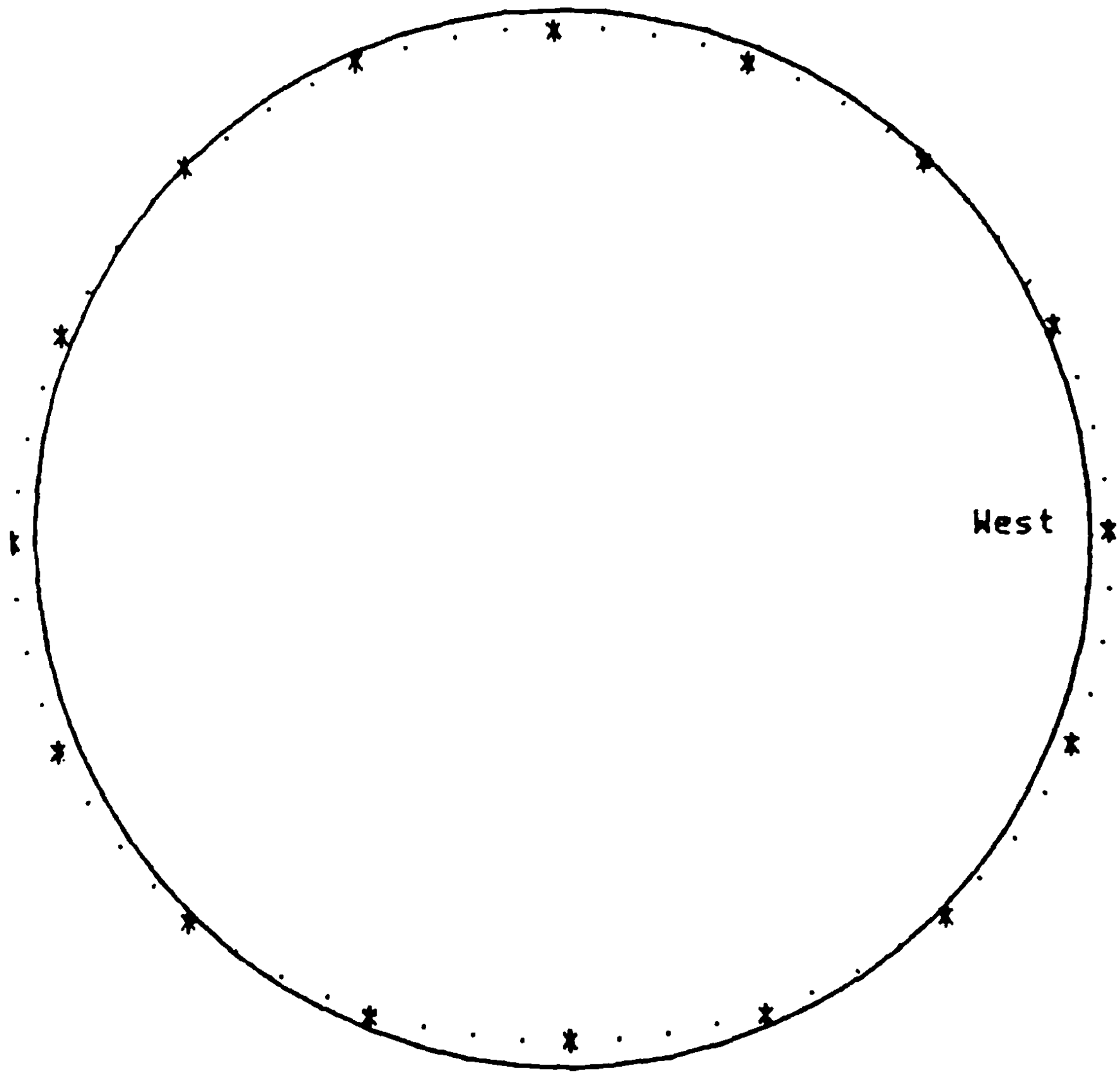

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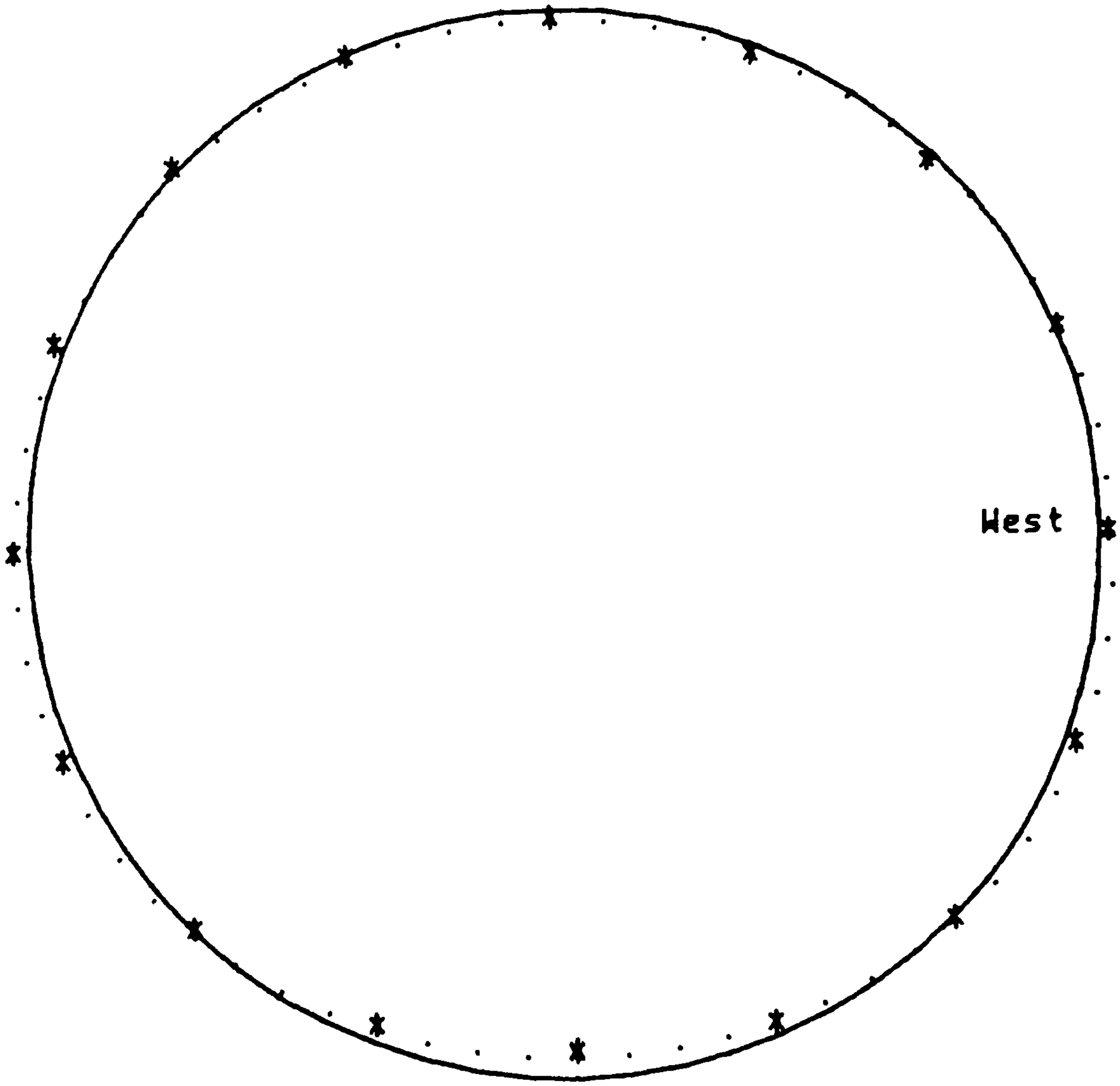

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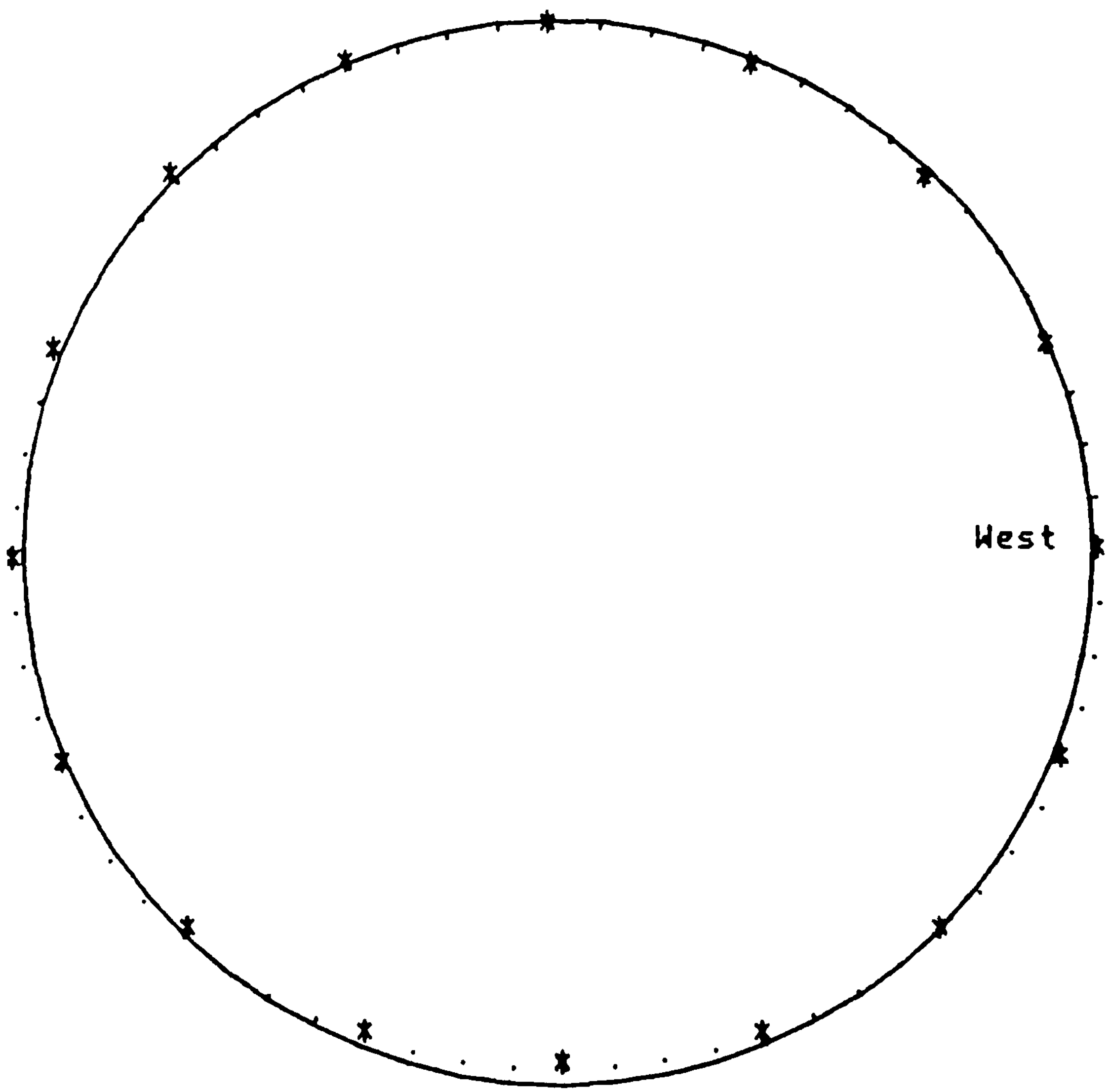

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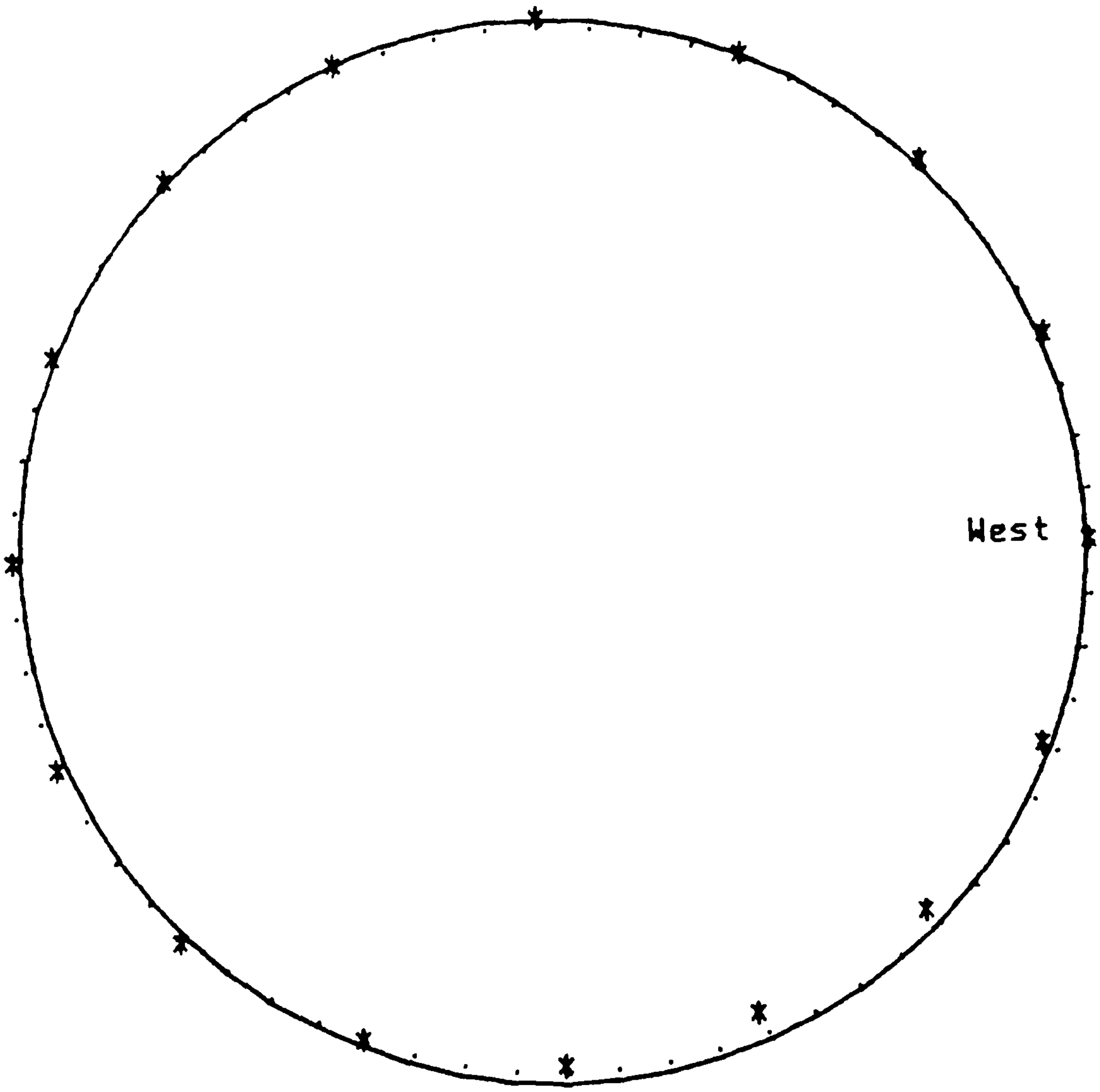

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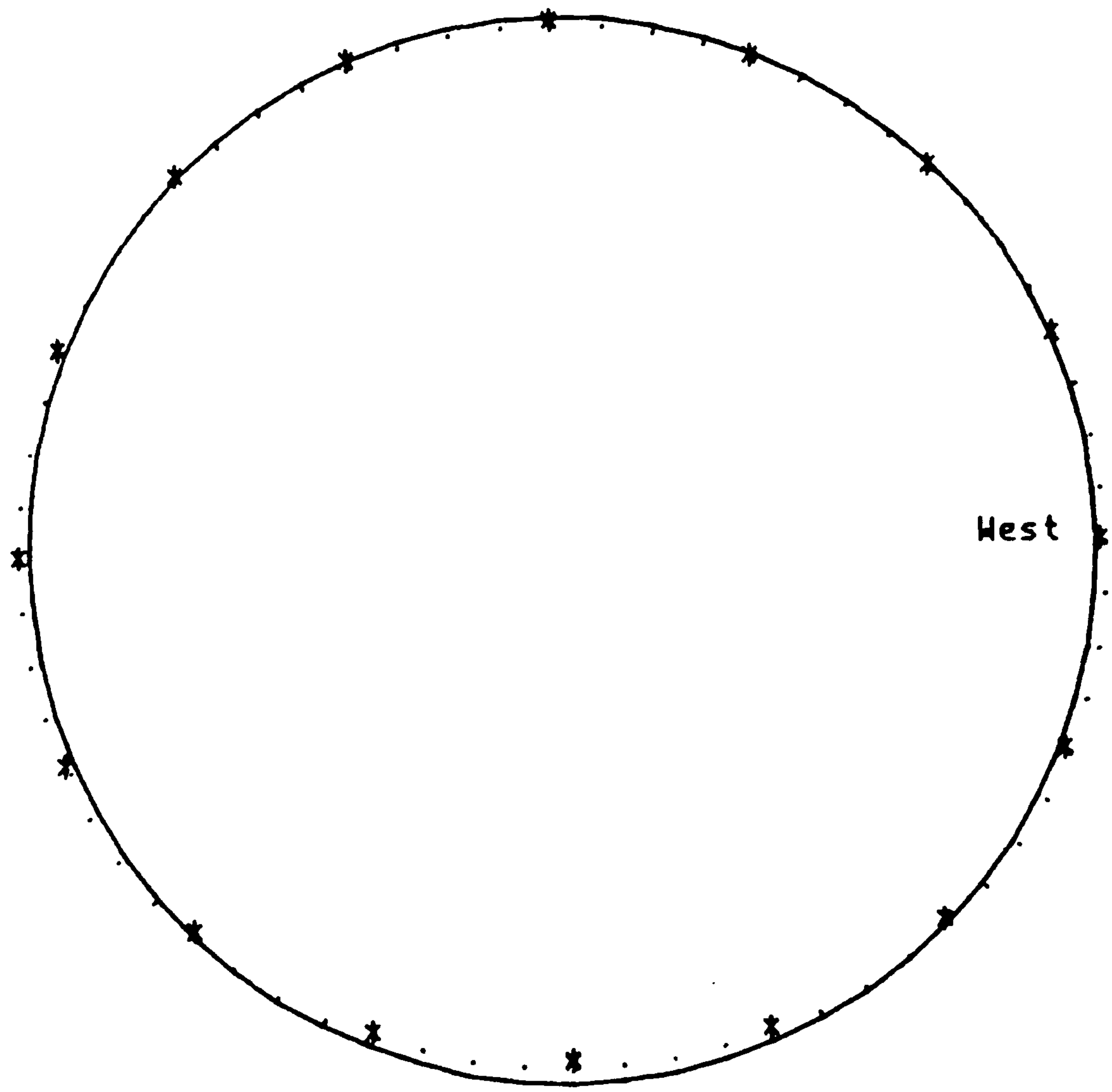

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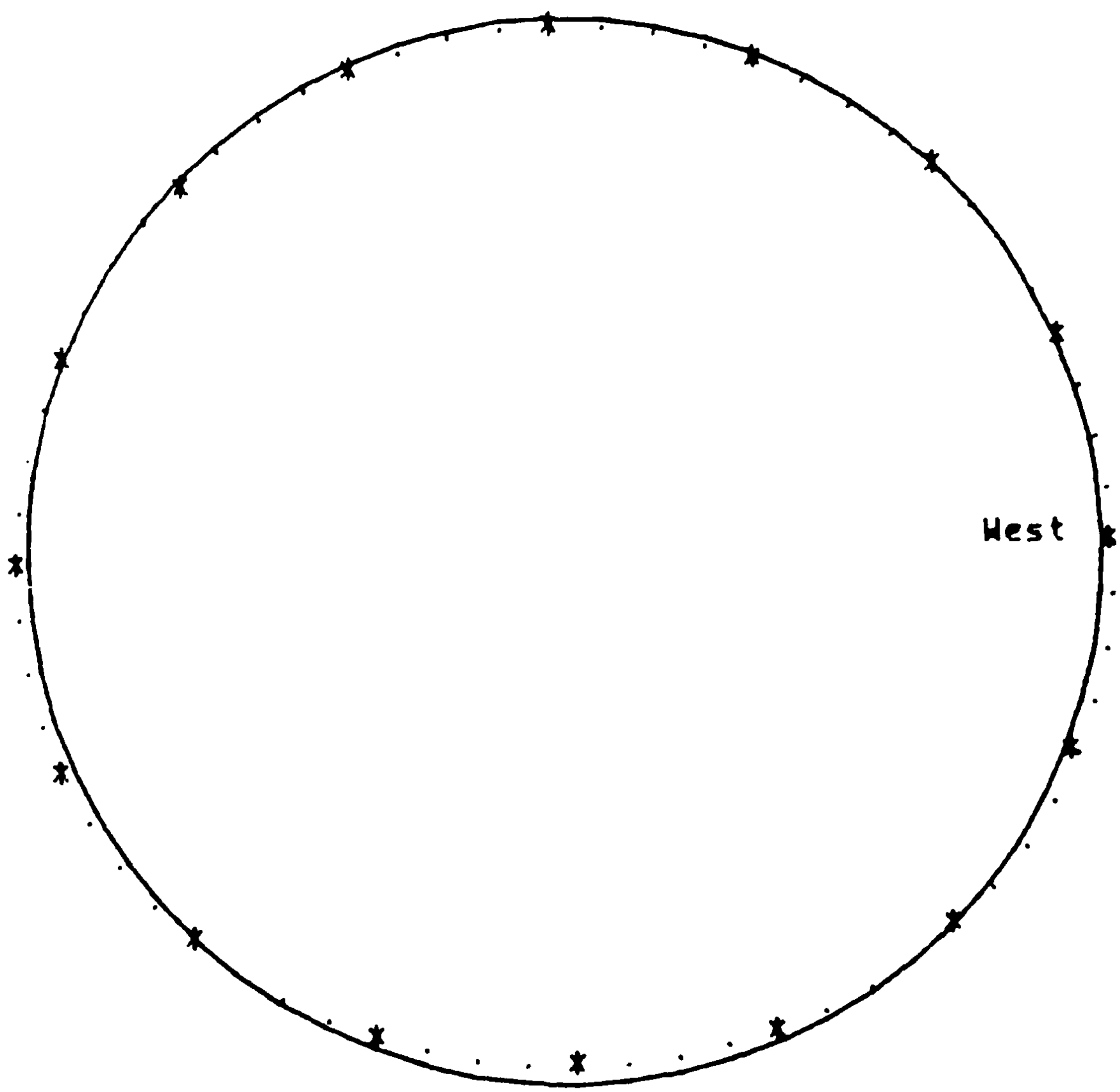

APPENDIX G

COMPUTER PROGRAM FOR RESISTANCE COMPUTATION

```
2000 REM DIMENSIONLESS SUBROUTINE FOR PLAIN CONCRETE MOMENT
2010 REM THRUST CURVATURE INTERACTION
2020 REM INPUTS: THRUST P      TENSILE STRENGTH T
2030 REM OUTPUT: MOMENT M      CURVATURE F
2040 REM SCRATCH: NEUTRAL AXIS DEPTH C
2050 REM          CUBIC EQN ITERATION C4,C6,P4,P5,P6
2060 IF P<2/3 THEN 2110
2070 C=1/SQR(3*(1-P))
2080 M=1/(12*C*C)
2090 F=1/C
2100 GO TO 2330
2110 IF P<(8-3*T*T)/(6*(2+T)) THEN 2160
2120 C=(3*(2-P)-SQR(9*(2-P)↑2-12))/2
2130 M=(2/C-2*C+C*C)/12
2140 F=1/C
2150 GO TO 2330
2160 DEF FNP(C)=T*(-T*C*C*C-12*C*C+18*C-6)/(12*(1-C)↑2)
2170 C4=0.5
2180 P4=FNP(C4)
2190 C6=2/(2+T)
2200 P6=FNP(C6)
2210 C=C4+(P-P4)*(C6-C4)/(P6-P4)
2220 P5=FNP(C)
2230 IF ABS(P5-P)<1.0E-3 THEN 2310
2240 IF P5<P THEN 2280
2250 P6=P5
2260 C6=C
2270 GO TO 2210
2280 P4=P5
2290 C4=C
2300 GO TO 2210
2310 M=T*(4-4*C-2*T*C↑3+T*C↑4)/(48*(1-C)↑2)
2320 F=T/(2*(1-C))
2330 RETURN
```

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Albert, Dick
15 Mile Road/Edison Corridor Sewer Tunnel Failure Study, Detroit Area, Michigan / by Dick Albert ... [et al.]. (Geotechnical Laboratory. U.S. Army Engineer Waterways Experiment / U.S. Army Engineer District, Detroit) ; prepared for Environmental Protection Agency, Region 5. -- Vicksburg, Miss. : U.S. Army Engineer Waterways Experiment Station; Springfield, Va. : available from NTIS, 1981.
185, [212] p. : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; GL-81-2)
Cover title.
"January 1981."
1. Field control tests (Soils). 2. Laboratory tests. 3. Structural behavior. 4. Tunnel failures. I. United States. Army Engineer Waterways Experiment Station. Geotechnical Laboratory. II. United States. Army. Corps of Engineers, Detroit District. III. United States. Environmental Protection Agency, Region 5. IV. Title. V. Series: Technical report (United States. Army Engineer Waterways Experiment Station); GL-81-2.
TA7.W34 no.GL-81-2

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